GEOTECHNICAL AND STRUCTURAL ASSESSMENT REPORT

FISHER BORDER WALL
Mission, Texas

Prepared for:

National Butterfly Center
Mission, Texas

Prepared by:

Millennium Engineers Group, Inc.
5804 N. Gumwood Avenue
Pharr, Texas 78577

August 28, 2020

MEG Project No. 01-20-29170

Millennium Engineers Group
MEG ENGINEERS
Consultants - Geotechnical - Testing

August 28, 2020

Mariana Trevino Wright
National Butterfly Center
3333 Butterfly Park Drive
Mission, Texas 78572
nbc@nationalbutterflycenter.org

Subject: **Geotechnical and Structural Assessment Report**
MEG Project No.: 01-20-29170
Fisher Border Wall
Mission, Texas

Dear Ms. Wright:

Millennium Engineers Group, Inc. is pleased to submit the enclosed geotechnical engineering report that was prepared for the above subject project. This report addresses the procedures and findings of our geotechnical engineering study. Our recommendations should be incorporated into the design and construction documents for the proposed development. Please consult with us, as needed, during the design and construction process.

We want to emphasize that our firm be retained to ensure that actual field conditions are those described in our geotechnical report. We cannot over emphasize the importance that all our recommendations presented in this report and/or addendums to this report be followed. We look forward to continuing our involvement in the project by providing construction monitoring in accordance with the report recommendations and materials testing services during construction. We strongly recommend that we be a part of the preconstruction meeting to address any specific issues that are pertinent to this project.

Thank you for the opportunity to be of service to you in this phase of the project and we would like the opportunity to assist you in the upcoming phases of the project. If you have any questions, please contact our office at the address, telephone, fax or electronic address listed below.

STATE OF TEXAS
ANDRES PALMA
115936
LICENSED
PROFESSIONAL ENGINEER

08/28/20

Cordially,
Millennium Engineers Group, Inc.
TBPE Firm No. F-3913

Andres Palma, P.E.
Geotechnical Engineer

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BORDER WALL REPORT, Mission Texas

1.0 INTRODUCTION

1.1 Background Information

Millennium Engineers Group, Inc. (MEG) was contracted by the National Butterfly Center to conduct an assessment of the current conditions as it relates to the geotechnical and structural of a recently constructed border wall in Mission Texas. The border wall is an 18 foot high galvanized steel bollard wall constructed on a poured in place concrete footing. The project also consists of a rigid concrete road approximately fifteen feet wide and located on the northern side of the fence and/or opposite side of the river. The wall is constructed on privately owned land and is parallel at approximately thirty five feet to the Rio Grande River bank on the United States landside and within the floodplain between Stations (RS) 172.5 to 175.5 and located upstream from the Anzalduas Dam in Hidalgo County, Texas *(Source: HEC RAS UNSTEADY FLOW HYDRAULIC MODEL by TGR Construction, Inc., refer to full report in the Appendix).* The site visit was conducted on August 3, 2020 at 10 am. The project was constructed by *Fisher Industries* and is approximately 3.0 miles in length.

1.2 Project Objectives

The purpose for this report was to perform a limited observation and assessment of the current conditions at the project site. The project has had erosion of the slope on the riverside and is a concern from our client. The project may have sensitive issues regarding many affected parties such as property owners upstream and downstream from the project. The items of concern to be observed included but not limited to; soil erosion on the river side of the border wall, concrete cracking, irregular concrete edging on the river side, and soil characteristics.

1.3 Documentation

As reference, the *International Boundary and Water Commission* (IBWC) (United States Section, El Paso, Texas) submitted to our client a report named *"Technical Memorandum, Mission*

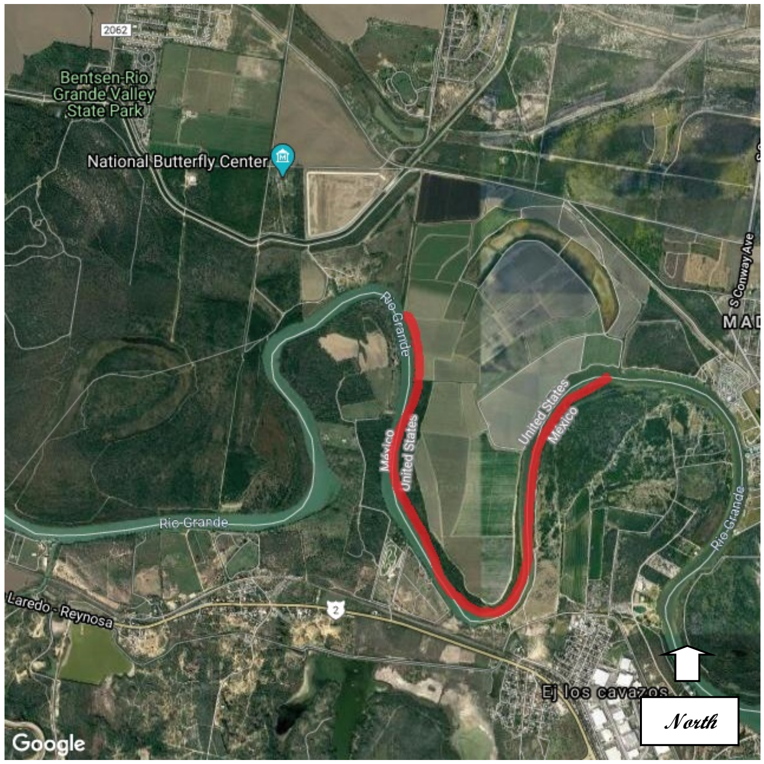
Texas Bollard Fence Hydraulics Analysis" dated March 2020; refer to report in the Appendix. As per this report, the IBWC's conclusion determined that the construction of the Fisher Wall Project was as follows:

> "There is significant deflection towards the U.S. approximately near the middle of the constructed bollard fence which needs to be mitigated. This can be done, for example, by including a gate in the location of the observed impact. If the bollard fence is extended, there may be hydraulic impact which needs to be verified through new hydraulic analysis. Although velocity distribution is similar in both existing and proposed condition models, the riverbank modified with 5h: 1v slope must be monitored for erosion occurrences and any observed erosion should be repaired by the proponent in a timely manner."

Additionally, the report titled *HEC RAS UNSTEADY FLOW HYDRAULIC MODEL by TGR Construction, Inc. dated December 31, 2019* was submitted to our client for review. As per this report comments and conclusion of the following is stated; 1) All existing vegetation on the bank in the area of this project will be replaced with deep rooted grass and all invasive vegetation (Carrizo Cane) is to be removed in effect eliminating caving banks as well as preventing trees and other vegetation from falling into the river channel during period , 2) A key part of this project is the stabilization and protection of the existing river bank requiring an ongoing maintenance program and 3) In situations where any erosion occurs it should be repaired in a timely manner and the area reinforced if necessary using riprap with an 18" to 36" diameter or an alternative geo-cell slope stabilization material. All these points, as a coordinated effort, will go a long way towards keeping the alignment of the river in the correct and mutually agreed upon location and is a key issue with regard to protecting the river and the overbank areas from further damage and degradation.

2.0 LOCATION OF PROJECT

The project wall was constructed along the bank of the floodplain on the United States side of the Rio Grande River south of Mission Texas and southwest of the intersection of Old Military Highway and South Conway Avenue. See Project Location Map below.



Project Location Map

3.0 SITE CONDITIONS

3.1 Description

Photographs submitted previous to our site visit showed how rainfall has created many large areas of erosion on the riverside of the wall. The observations of the structure began on the western side of the project. Many of the areas affected showed variations of widths and depths of the erosion cavities seen. These erosion cavities were as deep as 18-24 inches, see *Photo 1*.

Prior to our assessment, most erosion cavities on riverside of the wall had recently been back filled for approximately one mile starting on the western side of project. Bull dozer activity of refilling and grading existing erosion continued during our visit see *Photo 2*. The elevation difference between the bollard wall and the Rio Grande River as it appeared was about 10 to 15 feet, see *Photo 2.* Embankment soils appeared to be clayey sand to silty sand as filtered by touch by our geotechnical engineers.

During our observations and assessment, various measurements were taken to determine the depths of erosion cavities and void spaces under the concrete foundation of the bollard wall.

Photo 1 – Large Areas of Erosion

Photo 2 – Re-graded Embankment at start of inspection

Photo 3 – End of Re-grading of Embankment; approximately one mile east from the start of the wall alignment

4.0 OBSERVATIONS

4.1 Structural Conditions

The wall together with its foundation and the surrounding soil is sketched in *Figure 1* below. The foundation type of the wall is cast-in-place strip foundation. As in *Figure 1*, the distances from the edge to the center of the foundation were designated as ***B1*** (on the US landside) and ***B2*** (on the river side). ***B1*** was estimated to be about 4.5 feet and ***B2*** was measured to be between 3 feet 8 inches to 4 feet. The depth of the foundation, ***D***, was best estimated to be 2 feet. The distance of the wall to the nearest edge of the river is varied from 30 to 50 feet. The slope of the embankment at the riverside of the wall is varied from 3:1 to 5:1.

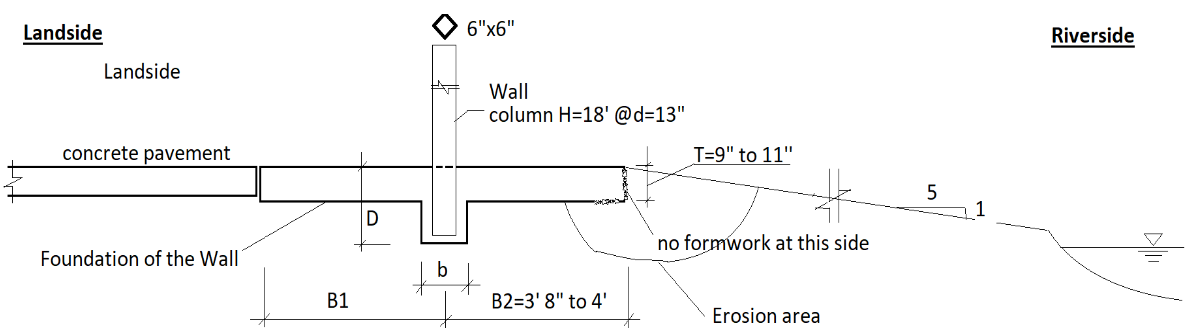


Figure 1: Typical Cross Section of the Fischer Bollard Wall

On the riverside, the thickness of the pour strip foundation, ***T***, was measured at various locations and range from 9 inches to 11 inches in depth. It is our assessment that the pour strip was placed directly on untreated soil. This makes the quality of the concrete at the bottom of the foundation lessen leading to the reduction of the effective thickness of the strip. The strip foundation was not poured using forms or edge restraint and appears to be continuous as no isolation joints were easily visualized, see *Photo 5*. On the river side, many places show signs of spalling at the very edge of the foundation. There was no opportunity to obtain dimensions on the land side.

Photo 4 - Typical Bollard Separation

The wall bollards are made of corrugated steel 6 inch x 6 inch pipes imbedded into the concrete footing. Their spacing's are approximately 13 inches center to center. The bollards are rotated 45 degrees to each other to have about a 5 inch clearance, see *Photo 4*. The bollards stand 18 feet tall from top of foundation. Since no construction plans were submitted to our client, the depth of footing dimensions shown on *Figure 1* can only be estimated based on our visual observations and our experience with border wall construction.

4.1.1 Concrete Quality

At the inspected locations, the bollard wall appears stable. Concrete paving is located on the landside of the bollard wall and does not show any signs of cracking, shrinkage, or deflection. The cracking of the wall foundation exhibited cracks less than 1/8 inch in width, see *Photo 6.*

Photo 5 – Measuring Soil Voids Under Foundation

Photo 6 – Concrete Shrinkage and Cracking Signs

The riverside of the concrete flatwork also showed cracking of less than 1/8 inch widths. Areas where erosion has taken place do not appear to have caused external visual damage to the bollard wall footing and poured strips. It appears no formwork was used on the riverside poured strips to help maintain a consistent straight edge and depth of concrete. Spalling and honeycombing were present in various locations of the riverside

poured strips. The concrete slump at these locations appeared to be on the drier side of the moisture range when the concrete strip was poured, see *Photos 7 and 8*.

4.1.2 Structural Deflection

At the time of inspections, there was no sign of structural deflection other than the cracks, however, there were indications that a potential risk is due to the soil eosin under the wall foundation.

Photo 7 – Spalled Concrete Edges of Pour Strips

4.1.3 Drainage

The drainage surface area of the concrete appears to slope from the landside to the riverside of the wall. This side of the bollard wall exhibits most, if not all, the erosion conditions.

4.1.4 Form work

It was evident the riverside foundation was poured without formwork. The concrete exhibited rough and segregated edging along the entirety of the foundation, see *Photos 7 and 8.* Most of the areas of segregated edging had spalling issues as well as exposed

aggregate indicating concrete with dry conditions of efflorescence. The thickness of this concrete edging was varying in depths, see *Photo 8.*

Photo 8 - No Form Work Utilized

4.2 Geotechnical Conditions

4.2.1 Slope embankment

The embankment area on the riverside of the wall appeared to be a 5:1 slope and is what was stated to our team. At time of our site visit, about one mile of the embankment had previously been re-graded by bulldozers. Most of the areas of the re-graded embankment did not have any grass germination and the invasive Carrizo Cane was growing back on the river bank, see *Photo 9.* The embankment on the river side appeared to be in stable condition.

4.2.2 Erosion of embankment

There are signs showing potential risks for the wall due to erosion. At various areas of the bollard wall, the soil under the wall foundation was eroded, which is visibly shown in *Photo 10*. Note that the time of the inspection occurred after *Hurricane Hanna*. Erosion of the embankment was 3 to 4 feet in depth in some areas; erosion width encountered was up to 2 to 3 feet. The actual working width of the pour strip foundation ***B,*** which is the sum of ***B1*** and ***B2***, see *Figure 1*, was reduced considerably. In some areas the pour strip working width was reduced by 30%, affecting the overall stability of the wall by allowing

soil to be eroded and loss of the lateral capacity of the soil in the areas where erosion occurred.

Photo 9 – Re-graded Embankment

Due to the erosion conditions, our technical concerns include:

- Center of the contact area of the foundation and the ground was moved toward the landside potentially causing an increase in the eccentricity,
- Bearing capacity of the pour strip foundation was reduced leading to a reduction in the factor of safety for bearing capacity,
- Reduction of the contact area between the foundation and the surrounding soil/ground can cause deformations, differential settlement of the foundation
- Shrinkage cracks are visible on top of the wall foundation; at the time of our site visit they are relatively small, however, these cracks may affect the performance of the foundation in the long term. This item should be reevaluated our time.
- The embankment should be sealed by a clay cap of impervious material to protect the existing embankment from erosion.
- Stone protection should be used to prevent erosion

Photo 10 - Erosions under Foundation

4.2.3 Native In-situ Material

The native in-situ material closer to the base of the embankment was silty sand or sandy silt. The material is very susceptible to erosion due to its lack of cohesion. This material is sensitive in nature to saturated conditions and has considerable change in reduced shear strength when the loose sand is in saturated conditions.

4.2.4 Embankment

The embankment material used in the construction of the slope was clayey sand (SC) to sandy clay (CL). This material is also susceptible to erosion due to its lower plasticity index. Embankment material is generally lean clay with a smaller percentage of sand, plasticity index typically ranging from 20 to 30 percent and material that will pass dispersion and crumb testing, see *Photo 11.* The material used in the construction of the embankment may not meet industry standards for embankments and or levees.

4.2.5 Impervious Clay Cap

No impervious clay cap or other forms of erosion control were encountered during the inspection. The soil type as installed has no erosion control and/or soil improvement. Native soil as installed will be challenging and prone to erosion effects in the event of saturated conditions, large rainfall events, or lack of maintenance.

Photo 11 - Silty-Sand Embankment Material

5.0 RECOMMENDATIONS

5.1 Embankment Improvements

The embankment cross section is likely to be susceptible to slow, long-term groundwater seepage throughout its width due to the nature of farm irrigation on the landside, the pressure head of the irrigation occurring at the landside, the height of water at the river level, and the wakes that occur from Border Patrol and boating equipment. Small boating equipment from the Border Patrol produced a wake of about 3 feet to 5 feet during the time of the inspection.

5.2 Consideration of River Elevation

The elevation difference between bollard wall foundation and the water level in the river is 10 -15 feet. This difference would not appear to be sufficient for any large rainfall event such as a 100 year storm or in drastic increases in the river's elevation when a discharge event occurs from northern (upstream) segments of the river and/or discharges from Mexico bodies of water.

5.3 Consideration of IBWC Noted Deflections

As per the *IBWC* report and their stated comments on the constructed bollard wall, the flow deflections as per their modeling hydraulic impacts for this 3 mile long bollard project were found to be minor. However, there is significant deflection towards the U.S. approximately near the middle of the constructed wall which needs to be mitigated.

5.4 Consideration of a Maintenance Program for Noted Deflection, Shrinkage Cracking and Erosion over time:

We recommend the following observations are performed to evaluate elevations, deflections, shrinkage cracking and erosion.

- Quarterly observations of the entire length of the project wall.
- Mitigate hydraulic impacts by including a gate in the location of the observed impact. If the bollard wall is extended, there may be hydraulic impact which needs to be verified through new hydraulic analysis (*Source: IBWC March 2020 report found in appendix).*
- Shrinkage cracking of concrete foundation should be closely monitored and mitigated as needed.
- Erosion impacts should be closely monitored. Proper construction of the embankment erosion can be mitigated as needed by utilizing methods of installation of a clay cap, stone caps and/or rip rap for surface level protection of erosion.

5.5 Evaluation for the Potential of Future of Loss of Land, Slope Stability by a Geotechnical Engineer after large upstream events or large rainfall events.

CONCLUSIONS

The geography at the wall's construction location in comparison to the river bend is not at a favorable location for long-term performance. Due to its location, geology, and topography there are several items that are factored into consideration by a Design Team.

Sediments, silts, sands are known to be encountered at various areas of river bends throughout the Rio Grande Valley. These river bends have naturally produced some adjacent properties at river bends to be sand and fill quarries due to their high sand content. Foundations that need to be placed on sandy soils when combined with lower groundwater table have often resulted in a decrease in soil strength thus requiring larger sizing of

foundation elements. The design documents were not allowed to be reviewed at the time of our site visit. The design of the wall, factors of safety and geotechnical conditions should be evaluated by a professional Engineer. Long-term concerns for the items observed during our site visit include Structural Performance, Global Stability of the Bollard Wall and Embankment, Erosion Control Measures and a suitable Maintenance Program.

Summary review and points stated by Mark Tompkins, P.E. and a Doctor in fluvial Geomorphologies Engineering based on aerial photographs of the vicinity of the Fischer Industries bollard fence project, ground photos, other documentation of the Rio Grande River, previously reviewed and conducted hydraulic analyses using the HEC-RAS model prepared by TGR Construction and the report titled "HEC-RAS UNSTEADY FLOW HYDRAULIC MODEL DETAILED SECTION AT MISSION TEXAS FOR BORDER FENCE INSTALLATION (found in the appendix of this report) is as follows:

- The 18 foot high bollard wall would change hydraulic sediment transport and debris transport conditions in and along the Rio Grande River adjacent to, upstream of, and downstream of the proposed project during future high flow events, and these hydraulic sediment transport and debris transport changes would detrimentally impact lands owned by the National Butterfly Center.

- Dr. Tompkins conducted additional hydraulic analyses that indicate the fence will either fail, damage adjacent lands, or both. Dr. Tompkins also expects with a reasonable degree of scientific certainly that the bank erosion and scour depths that occur with a massive increase in shear stress imparted by the flowing water on the river bank will undermine the relatively shallow foundation of the bollard fence and cause it to fail.

- Ground photos show extensive growth of vegetation on the previously cleared river bank and the lack of a maintenance plan, the vegetation will continue to grow and within a year or two will effectively clog the openings of the bollards which hydraulic modeling clearly shows this event will redirect flows and exacerbate erosion and scour upstream along and downstream of the fence project thus creating scattered flow obstructions with damaging and unpredictable impacts.

ADDITIONAL SITE PHOTOS

Long Erosion Crevices from Wall to River Bank

Elevation Profile of Embankment looking Westerly (Notice the Re-growth of the Carrizo Cane) (Minimal grass seeding germination) Large erosion voids

APPENDICES

1. Report (Copy)
   INTERNATIONAL BOUNDARY & WATER COMMISSION,
   Technical Memorandum, Mission, Texas Bollard Fence Hydraulic Analysis
   March 2020

2. Report (Copy)
   HEC RAS UNSTEADY FLOW HYDRAULIC MODEL
   Detailed Section at Mission, Texas for Border Fence Installation
   Submitted by TGR Construction, Inc.
   December 31, 2019

INTERNATIONAL BOUNDARY & WATER COMMISSION

United States Section, El Paso, Texas

March 2020

TECHNICAL MEMORANDUM

MISSION TEXAS BOLLARD FENCE

HYDRAULIC ANALYSIS

Hidalgo County, Texas

March 2020

TECHNICAL MEMORANDUM

MISSION TEXAS BOLLARD FENCE
HYDRAULIC ANALYSIS

Hidalgo County, Texas

INTERNATIONAL BOUNDARY & WATER COMMISSION
United States Section
El Paso, Texas

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1.0 INTRODUCTION

The purpose of this Hydraulic Report is to assess the hydraulic impacts of an approximately 3-mile long bollard fence constructed by a private firm Fisher Industries in early 2020 on private land along the bank of the Rio Grande floodplain in Hidalgo County, Texas, and to determine if these impacts are minor and consistent with Article IV B(1) of the 1970 Boundary Treaty. To ensure impacts are minor, the U.S. Section of the International Boundary and Water Commission (USIBWC) has criteria stating that the design flood Water Surface Elevations (WSE) for the proposed (with project/fence) condition shall not increase more than 6-inches in rural areas or 3-inches in urban areas compared to the existing condition WSE and have no more than +5% increase in flow deflection towards either the U.S. or to Mexico. In areas where levees exist, there shall be no WSE increase as such increase would decrease the existing freeboard requirements by the U.S. Federal Emergency Management Agency (FEMA) regulation (44CFR 65.10). Adherence to these regulations is required for levees to be accredited by FEMA and remove areas on the landside from the floodplain resulting in residents not having to purchase flood insurance.

The U.S. Section had developed a hydraulic modeling methodology for a detailed analysis of the hydraulic impacts. This methodology was shared with the project proponent, who conducted the analysis and submitted a report (Fisher, 2020). This report documents additional analysis conducted by the USIBWC to analyze the hydraulic impacts.

Figure 1: Area map of Hidalgo County and Tamaulipas State at Rio Grande/Rio Bravo

2.0 LOCATION

The bollard fence was constructed by the private firm Fisher Industries along the bank of the floodplain on the U.S. side of the Rio Grande upstream from the Anzalduas Dam in Hidalgo County,

Texas. The exact project location starts at 3.36 miles upstream of Anzalduas Dam and extends to 6.36 miles upstream. Therefore, the length of the bollard fence is approximately three (3) miles. The fence alignment runs parallel to the river with an offset of 35 feet from the bank. The location is shown with the green place-mark in **Figure 1**.

Figure 2 shows the exact location of the bollard fence close to the left bank (looking downstream) of the Rio Grande. This project falls approximately within the following overall limits: Upstream (26^0 10' 15.90" N and 98^0 21' 28.21" W), and downstream end (26^o 10'1.80" N and 98^0 20' 30.84" W).

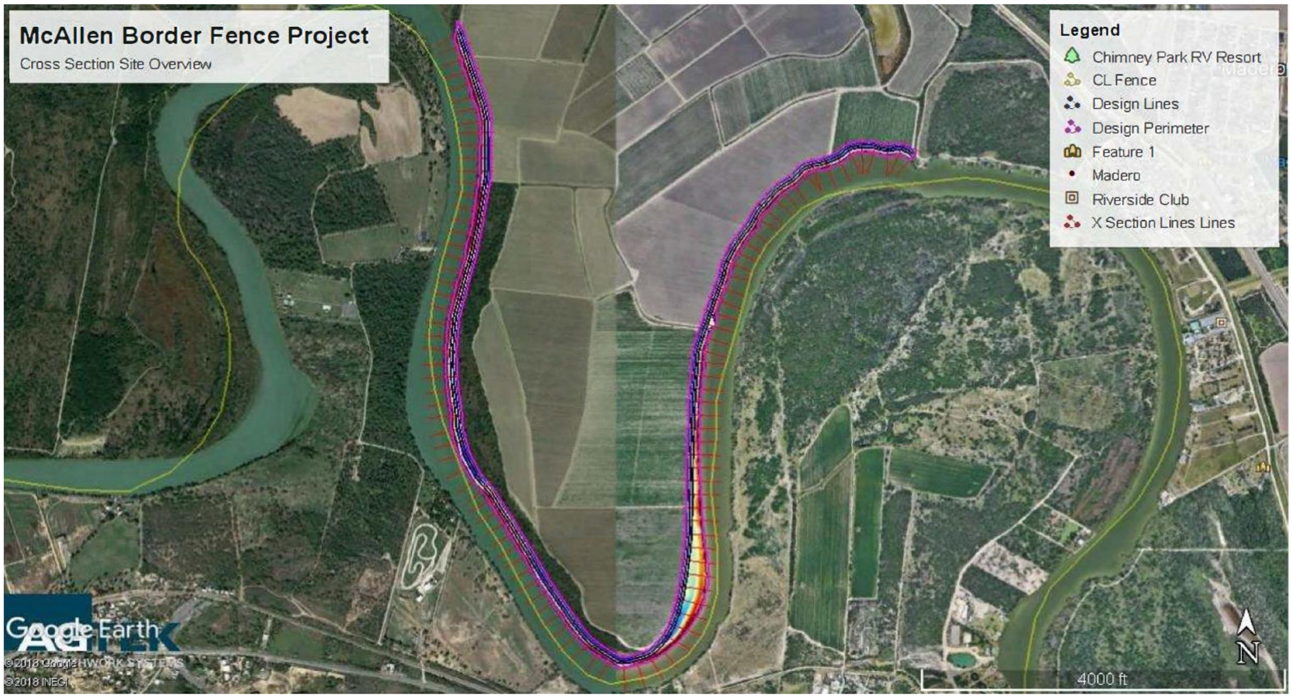


Figure 2: Bollard Fence Location on the Left Bank of Rio Grande (US Side)

3.0 HYDRAULIC ANALYSIS

Hydraulic modeling was conducted to assess the hydraulic impacts of the constructed bollard fence on the Rio Grande floodplain. The hydraulic impacts were evaluated using the design flow of 235,000 cfs for this reach of the Rio Grande (**Figure 3**). The hydraulic model was developed using the U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center - River Analysis System (HEC-RAS) software Version 5.0.7. A 1D/2D HEC-RAS model developed by Fisher Industries was modified to calibrate the model. The calibrated model was then used for the existing and proposed conditions modeling to assess the hydraulic impacts.

The HEC-RAS software supports simultaneous 1D channel and 2D floodplain components in a single model, performing calculation iterations between 1D and 2D areas. The 1D and 2D components are connected using lateral structures. These structures use the weir equation to exchange flows between 1D component and 2D mesh. A total of 16 lateral structures was used to exchange flows between river channel and floodplains.

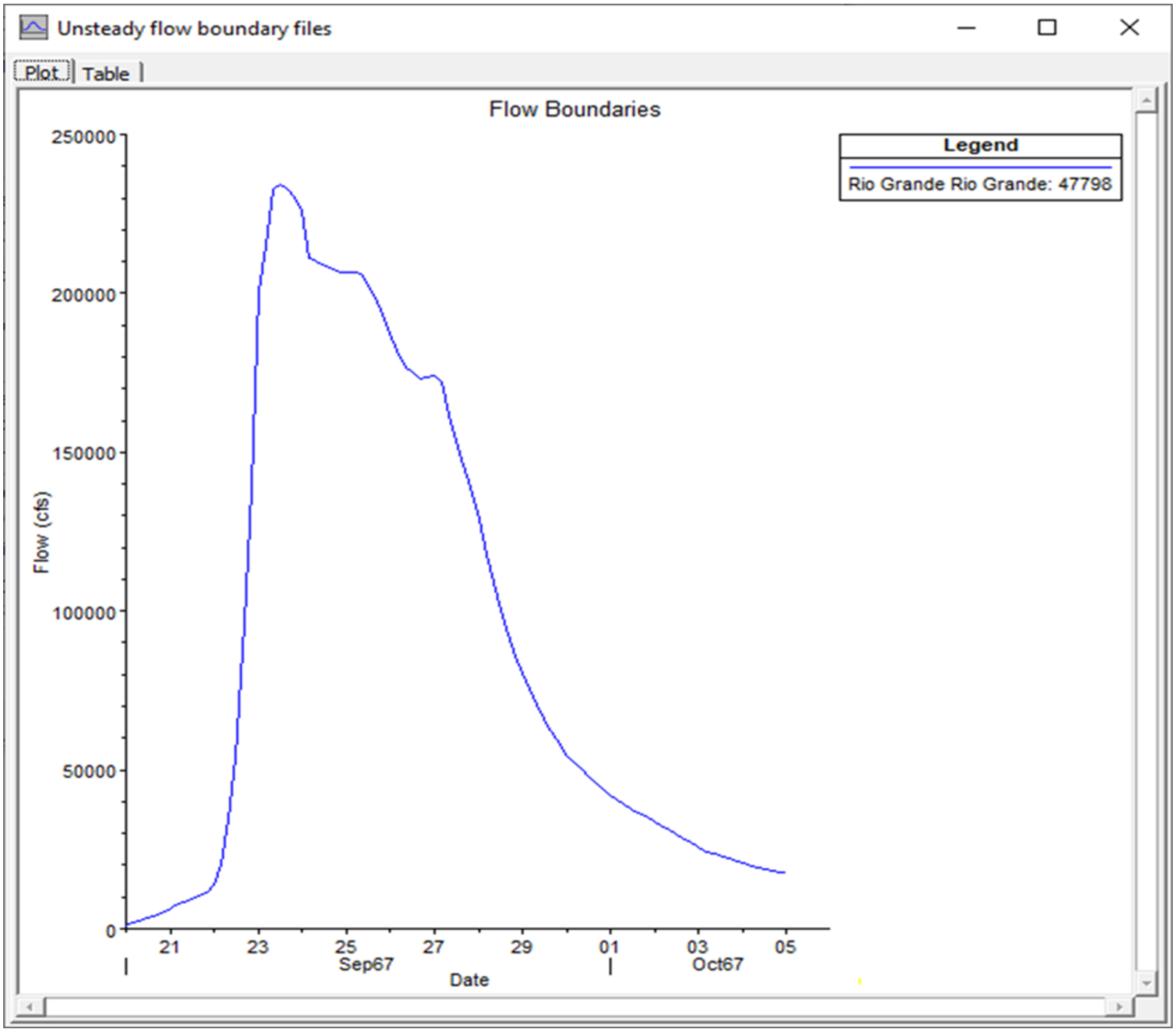


Figure 3: Design Flow Hydrograph with Peak of 235,000 cfs

4.0 INPUT DATA

4.1 Topographic/Bathymetric Data

LiDAR data (2011) was used to model the terrain. Surveyed cross-section by Fisher Industries, and cross-sections from the S&B (2008) model were used to construct the 1D channel cross-section at every 500 feet for the full length of the project site. Using channel geometry data and LiDAR dataset, a single terrain was created by the Fisher Industries. The terrain model developed by Fisher Industries was used to complete this hydraulic analysis. The horizontal datum is NAD 1983, State Plane Texas South 4205 Feet, and the vertical datum is NAVD88.

4.2 Manning's Roughness Values

Manning's roughness values were refined during calibration with observed stage near the downstream of the project site. Manning's roughness values for different regions of the modeled area are shown in Table 1.

4.3 Boundary Conditions

Boundary conditions were applied at the upstream project extent (RS 47798) with an inflow hydrograph with peak flow equal to the design flow of 235,000 cfs with hydrograph shape of the Hurricane Beulah of September 1967. The downstream boundary condition was normal depth with slope 0.01 foot/foot.

5.0 MODEL CALIBRATION

A detailed model calibration was conducted in this study. There are no measured water surface elevations within the project reach for past major floods. The nearest gaging station is at Los Ebanos, Texas, about 24 miles upstream from the project site. However, this gaging station malfunctioned during Hurricane Beulah and there was no recorded data once the peak flow was observed. Hurricane Beulah was recorded at the gaging station located in Rio Grande City which is about 57 miles upstream from the project site. The peak discharge of 220,000 cfs was recorded at midnight of September 22nd-23rd, 1967 at this gaging station.

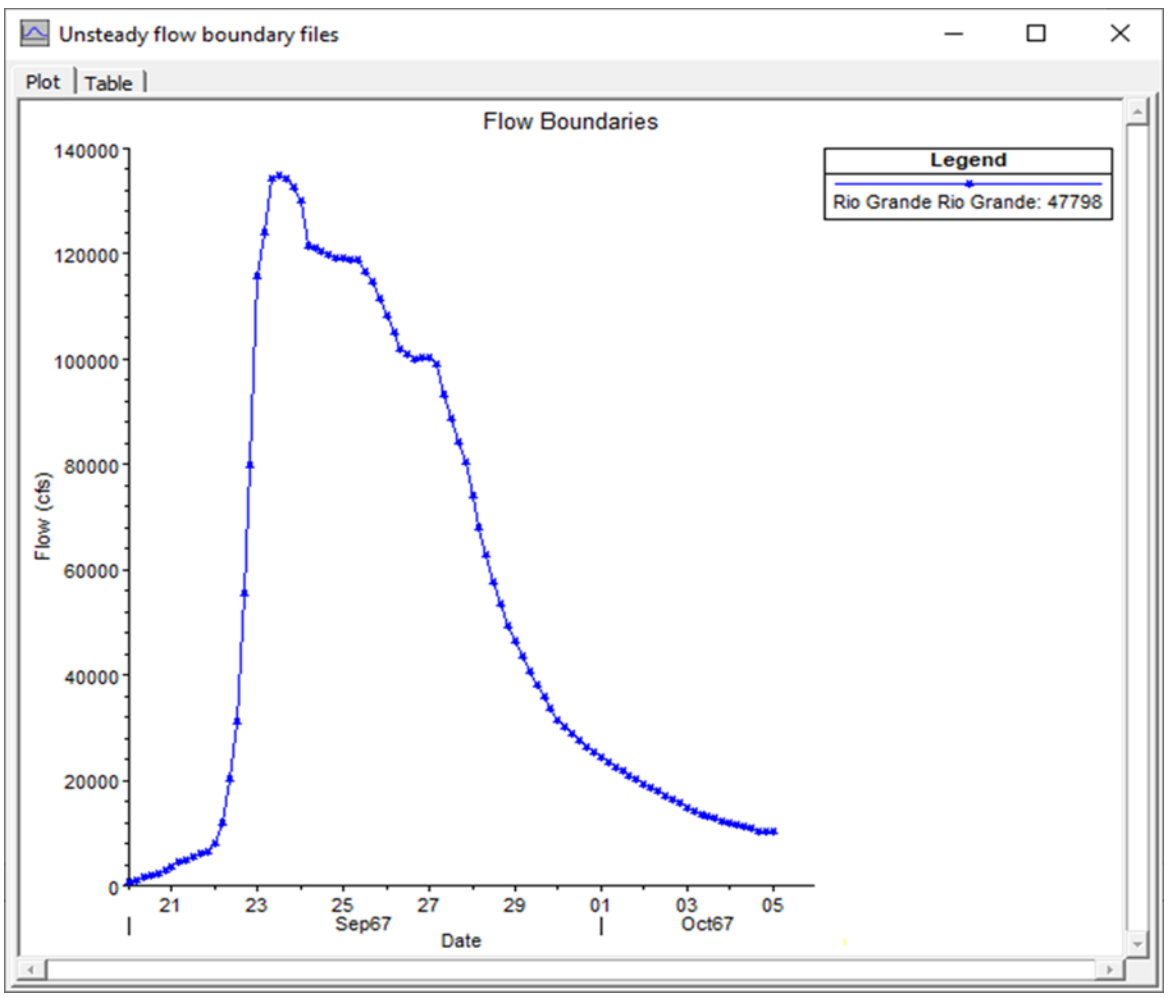


Figure 4: Hurricane Beulah Hydrograph (September 1967) at Half-Mile Downstream of Anzalduas Dam

During Hurricane Beulah of 1967, there was no flow diversion at Banker weir. Instead, flood waters were diverted from the Rio Grande through the Mission Inlet, located approximately 6.5 miles upstream of Anzalduas Dam. The remaining flow continues downstream. On September 24, 1967, the gage about 0.5-mile downstream of Anzalduas Dam had a maximum gage height of 112.82 feet (NAVD 88) with a peak flow of 131,000 cfs. Since there is no full hydrograph for the event, a hydrograph for model calibration was designed with a peak flow of 135,000 cfs with the shape of hydrograph adopted from Hurricane Beulah hydrograph observed at gaging station of Rio Grande City, Texas. **Figure 4** shows the hydrograph used for model calibration.

Several calibration runs were conducted to match the WSE upstream of Anzalduas Dam extrapolated from the value measured at the gage 0.5 mile downstream of the dam. The finalized roughness coefficient values based on calibration model run are shown in **Figure 5.** Channel roughness value is 0.035 with 0.04 for both overbanks. Floodplain comprises of row crop (corn), forest and cleared area. Roughness for various forest areas vary from 0.08 to 0.3, including some areas with 0.10. Roughness values have to be increased to 0.3 at the downstream areas to match observed WSE.

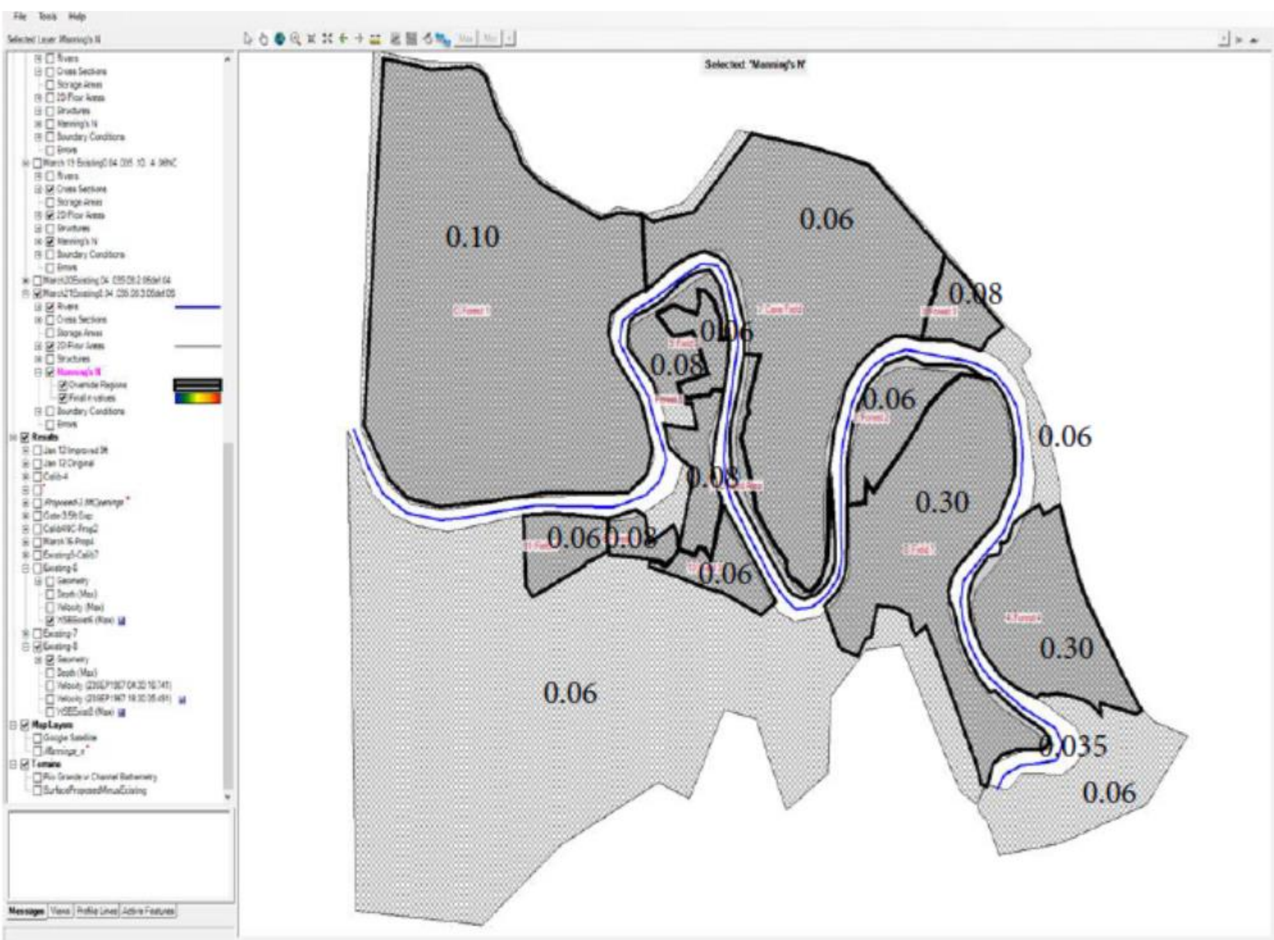


Figure 5: Final Manning's Roughness Values after Model Calibration

Based on 1D steady state S&B (2008) model, the WSE slope is about 0.000331 near the Anzalduas Dam. At this WSE slope, a point 445 feet upstream of the dam would have a WSE of 1.02 feet more than the WSE measured at the gage located 0.5 mile downstream of the dam. Thus, the peak measured WSE at 445 feet upstream of the dam would be 113.84 (112.82+1.02) feet. The model has simulated 113.45 feet WSE at this point which is very close to the observed WSE (113.84 feet). Maximum WSE obtained during calibration run is provided in **Appendix A**.

Channel	0.035	Overbanks	0.04
Cane Field	0.06	Forest 1	0.10
Forest 2	0.06	Forest 3	0.08
Forest 4	0.30	Forest 5	0.08
Forest 6	0.08	Field 1	0.30
Field 2	0.06	Field 3	0.06
Field 4	0.06		

Table 1: Manning's Roughness Values used in Simulation

5.0 EXISTING CONDITION MODEL

The existing condition model was used as a baseline to compare with the proposed condition model results and evaluate the hydraulic impacts.

The existing condition model includes:

- 2D computational grid with 25-foot grid cell size for the left and right overbanks
- 1D channel river reach
- 1D/2D lateral weir structures (16 of them) for flow movement from channel to overbanks
- downstream Anzalduas Dam as an inline structure

The inflow hydrograph with peak flow of 235,000 cfs and boundary conditions are as described above.

6.0 PROPOSED CONDITION MODEL

The proposed condition model for this project includes the constructed 18-foot high bollard fence along the U. S. side bank of the Rio Grande. The bollard wall consists of 6-inch square bollards with a 4-inch clearance between bollards. The bollards are rotated 45-degrees to have 5-inch clearance, as shown in **Figure 6**. For the proposed fence condition, to account for debris blockage during the design flood in the model, the clear space between adjacent bollards is reduced by 30%. That distance is added to the bollard widths to keep the center to center spacing between adjacent bollards the same. Therefore, in the 45-degrees rotated configuration, the open space becomes 3.5 inches and the bollard width becomes 7.5 inches (increased from 6 inches).

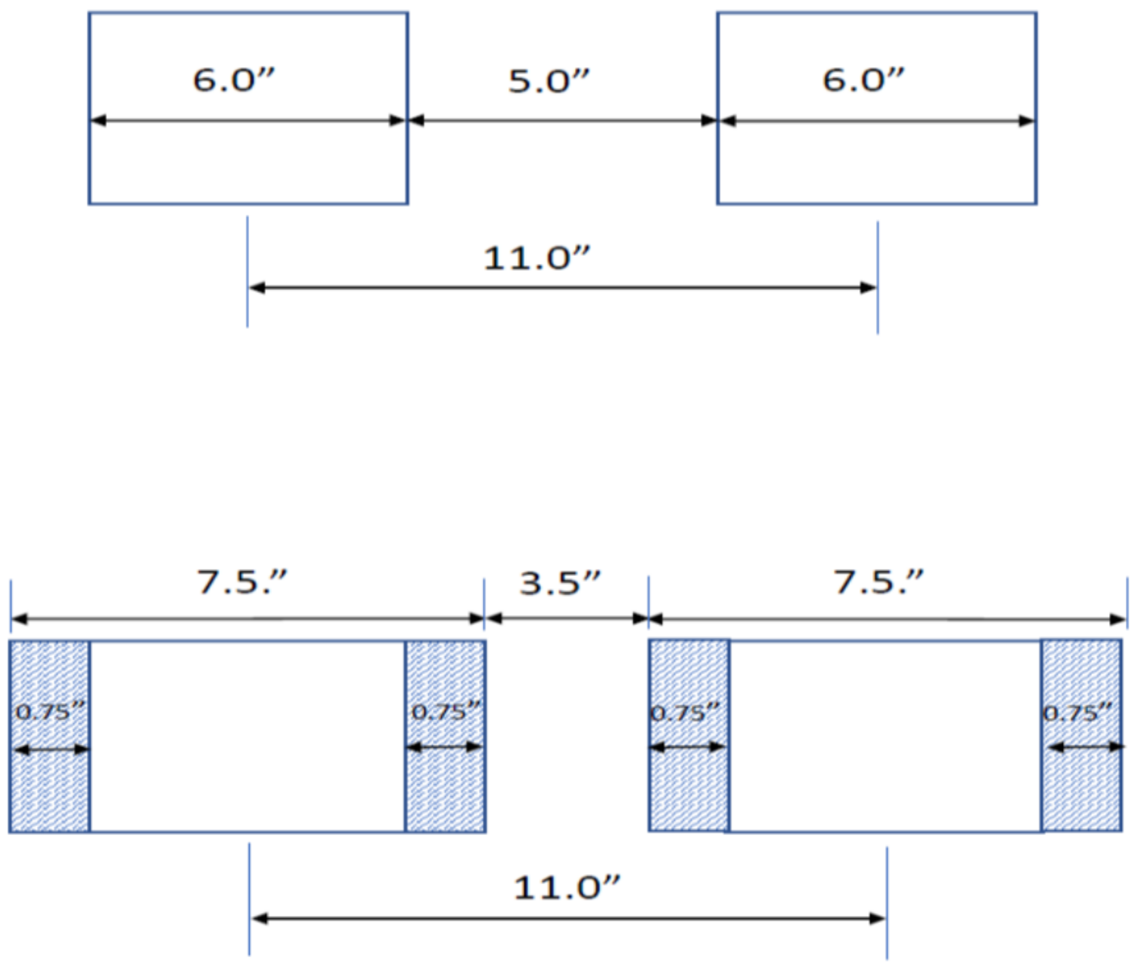


Figure 6: Regular Bollard Configuration (Top), Bollard Configuration with 30% Debris (Bottom)

The proposed condition model includes the elements in the existing condition model and:

- 18-foot tall bollard modeled as 18 feet gate with 3.5 feet width
- 2D left floodplain with 10 lateral structures, while 2D right floodplain with 8 lateral structures
- Bollard wall modeled with 59 gate groups with 1427 identical gate openings

Eighteen (18) lateral weir structures are used to connect the 1D channel with 2D left (USA) and right (Mexico) overbanks. The constructed bollard fence along the Rio Grande’s left bank (35 feet offset) is modeled as a lateral gate on top of the 1D/2D weir structure as shown in **Figure 7**. To avoid having a large number of lateral structures to represent the bollard fence in the model, adjacent bollards were combined. The spacings between the bollards were also similarly combined. Twelve (12) adjacent bollards were combined. Therefore, using the multiplier factor of 12 resulted in a bollard thickness of 7.5 feet and spacing of 3.5 feet. This also took into account the 30% blockage.

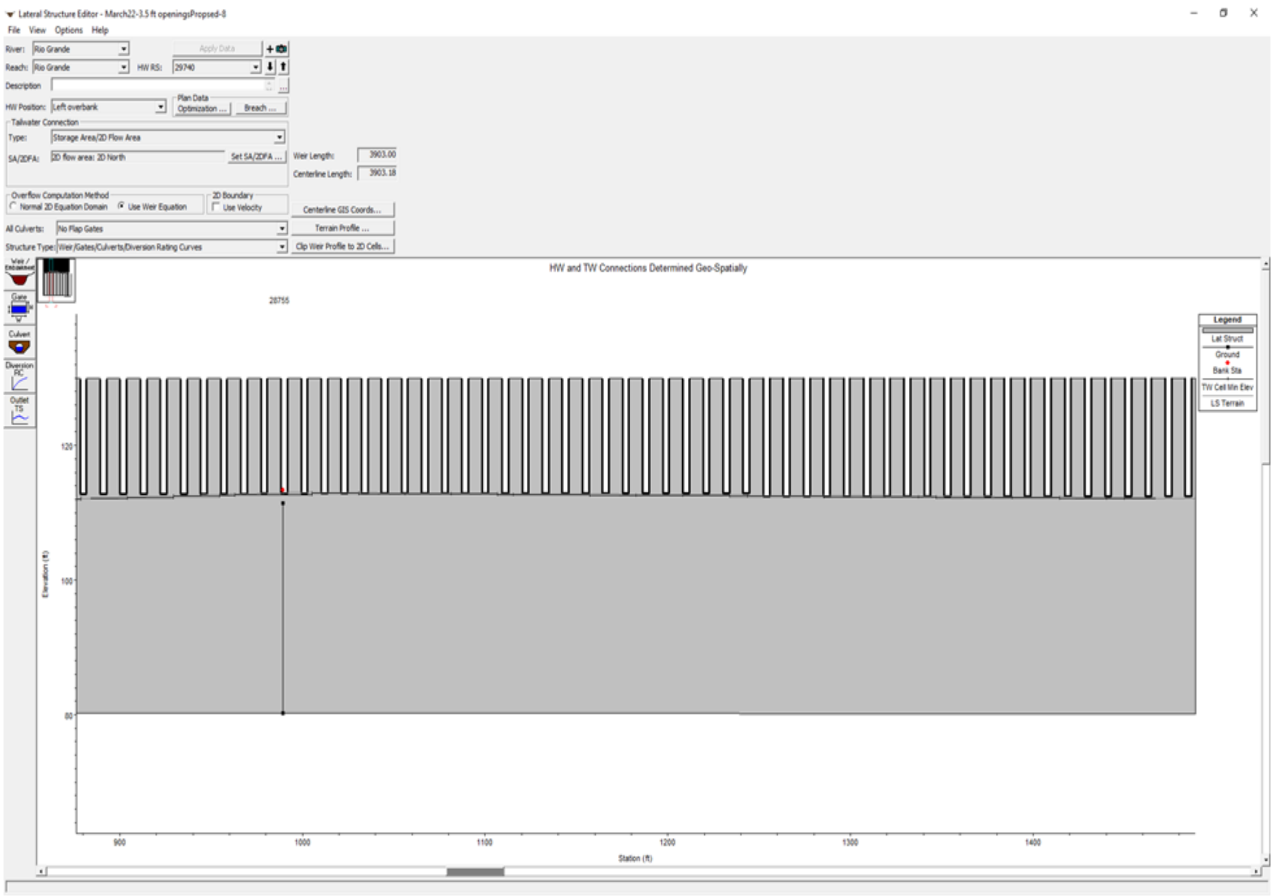


Figure 7: 18 Feet High 3.5 Feet Wide Gates (Gaps Between Bollards) With 7.5 Feet Bollard Wall

7.0 EVALUATION OF HEC-RAS OUTPUT

The USIBWC levee at the project site was designed and constructed to have three (3) feet of freeboard above the design flood WSE as estimated by the USIBWC (2003) model. The constructed bollard wall falls between levee stations 130+00 and 190+00.00 (**Figure 8**). WSEs from both existing and proposed 1D/2D models are compared with design flood WSE values to determine if levee freeboard is encroached. Existing condition model WSE values are close to the design flood WSE values in the USIBWC (2003) model. In the Fisher Industries modeling, the WSE values obtained were two (2) to three (3) feet lower than the WSE values in S&B (2008). The more detailed calibration conducted in this study resulted in a better match with the design flood WSE values from USIBWC (2003) and S&B (2008) models.

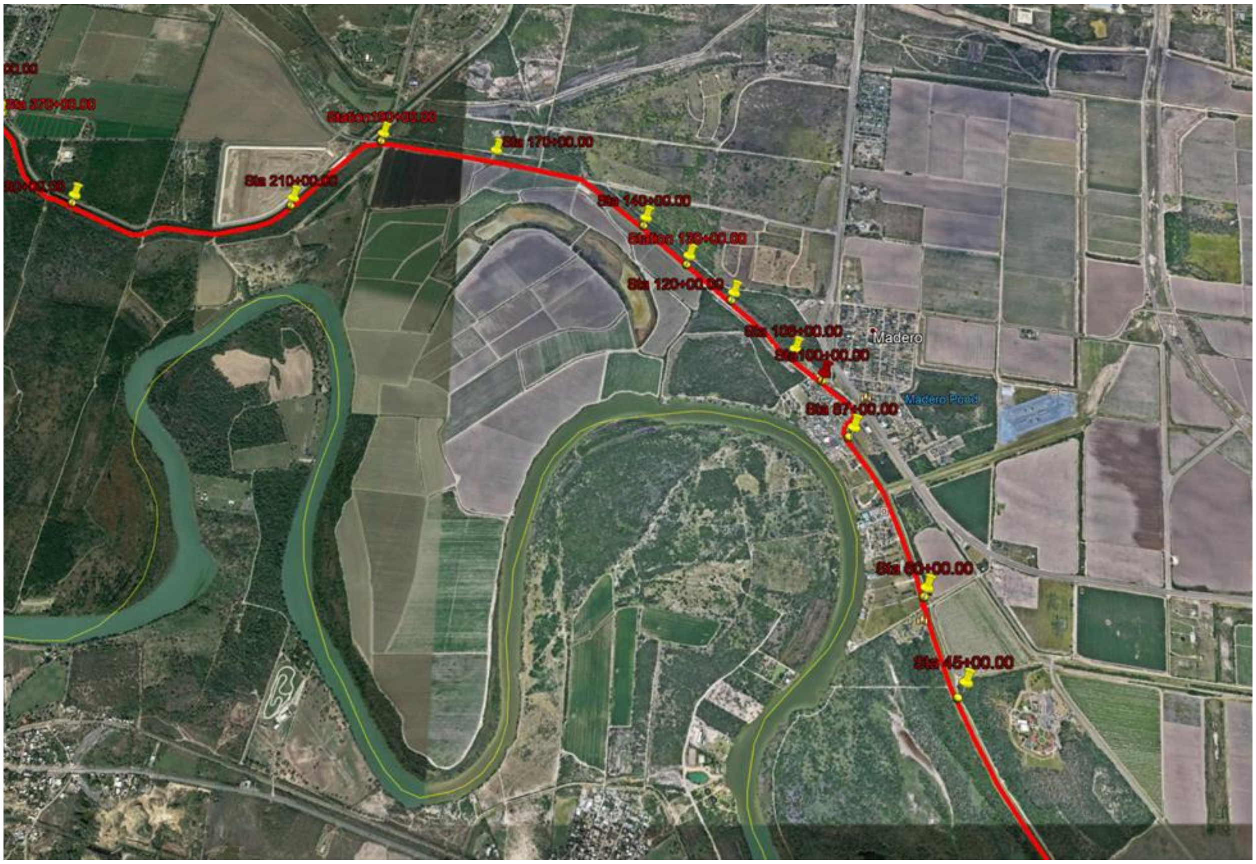


Figure 8: USIBWC Levee (marked red) at Project Site

Levee Station	Design Flood Elevation, ft (USIBWC 2003)	Existing Model WSE, ft	Proposed Model WSE, ft
45+00.00	123.23	123.13	123.30
60+00.00	124.39	124.79	124.79
87+00.00	126.48	126.35	126.34
120+00.00	127.18	127.55	127.19
130+00.00	127.35	127.70	127.27
170+00.00	128.00	127.82	127.40
190+00.00	128.31	127.89	127.44
250+00.00	129.45	128.87	128.45

Table 2: Water Surface Elevation Comparison

Maximum water surface elevations for the entire simulation period for both existing and proposed condition models are shown in **Appendix B**. With bollard fence placed in the floodplain close to the riverbank, it was expected that proposed condition model WSEs would be higher than the existing WSEs in all areas. But based on model simulation, proposed condition model WSE values are slightly lower than the existing condition WSE values at most locations (**Table 2**). This can be explained through flow patterns for the two conditions in the project site. Initially, the bollard fence provides resistance to flows and the flow is diverted around the wall as shown in **Figure C-1** through **Figure C-3** in **Appendix C**. As time passes, the flow pattern for proposed condition model becomes similar to that of the existing condition model (**Figure C-4** through **Figure C-6**). Flows in proposed condition model pass through the bollard fence gaps (3.5 feet) with slightly higher velocity than the existing

condition model where there is no flow resistance due to the fence. Thus, this slight increase in velocity can result in slightly lower WSE in the proposed condition model. Velocity distribution for both existing and proposed condition models are shown in **Appendix C**.

Although velocity distribution is similar in both existing and proposed condition models, the river bank modified with 5H:1V slope has to be monitored for erosion occurrences and any observed erosion should be repaired by the proponent in a timely manner.

8.0 ANALYSIS OF DEFLECTION CALCULATIONS

Flow deflections were evaluated at a limited number of cross sections. Profile lines for flow deflection calculations developed by Fisher Industries were used. The deflection calculations were repeated using the modeling results from this study. The highest flow deflection is 10.32% towards the U.S. at profile line 24260. **Figure 9** shows the deflection values to both the U.S. and Mexico at the location indicated.

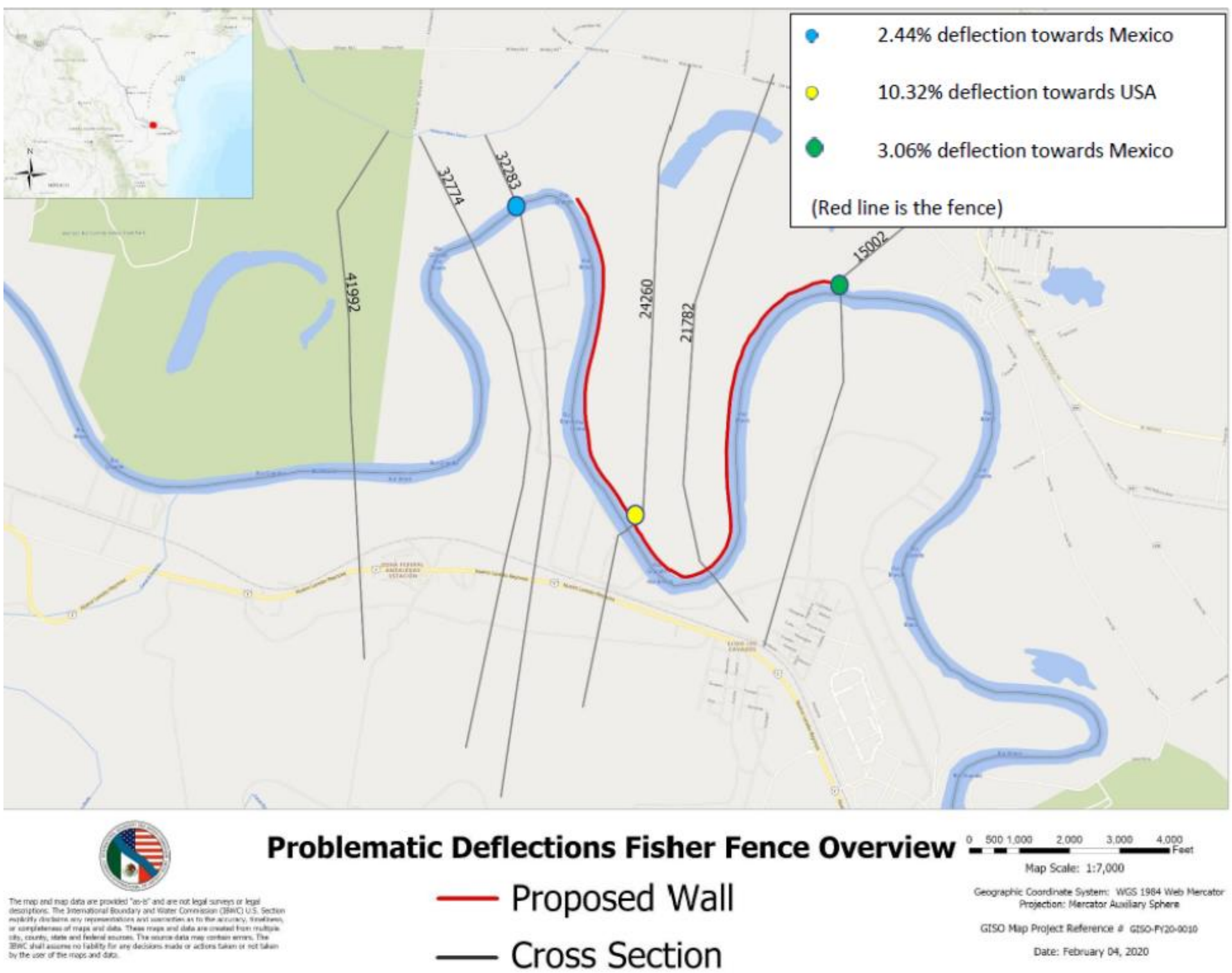


Figure 9: Flow Deflection along Selected Profile Lines at the Project Site

The hydraulic impacts for this 3-mile long bollard fence project were found to be minor. WSE elevations were found to be mostly lower in the proposed condition than in the existing condition, and therefore did not decrease existing levee freeboard. Percent deflection calculations exceeded the

+5% threshold of the USIBWC at one location in a limited number of cross sections analyzed. Flow deflections at other locations appears to be minor because flow is diverted to the left bank to the U.S. side by the bollard fence alignment. Also, the site is within the backwater area of Anzalduas Dam, resulting in lower flow velocities in the floodplain.

12.0 CONCLUSIONS

There is significant deflection towards the U.S. approximately near the middle of the constructed bollard fence which needs to be mitigated. This can be done, for example, by including a gate in the location of the observed impact. If the bollard fence is extended, there may be hydraulic impact which needs to be verified through new hydraulic analysis.

Although velocity distribution is similar in both existing and proposed condition models, the riverbank modified with 5H:1V slope must be monitored for erosion occurrences and any observed erosion should be repaired by the proponent in a timely manner.

11.0 REFERENCES

1. HEC-RAS River Analysis System, US Army Corps of Engineers Hydrologic Engineering Center, User's Manual, Version 5.0, February 2016.
2. USIBWC (2003), Hydraulic Model of the Rio Grande and Floodways within the Lower Rio Grande Flood Control Project, dated June 2003.
3. S&B Infrastructure (2008), Lower Rio Grande Flood Control Project HEC-RAS Hydraulic Model Update and Validation (Penitas to River Mile 28 and Off-River Floodways in Texas and Mexico), dated July 2008.
4. USIBWC (2020), Two-Dimensional Hydraulic Modeling Methodology.

Appendix A

Screen shots of Hurricane Beulah Hydrograph September 1967 Calibration

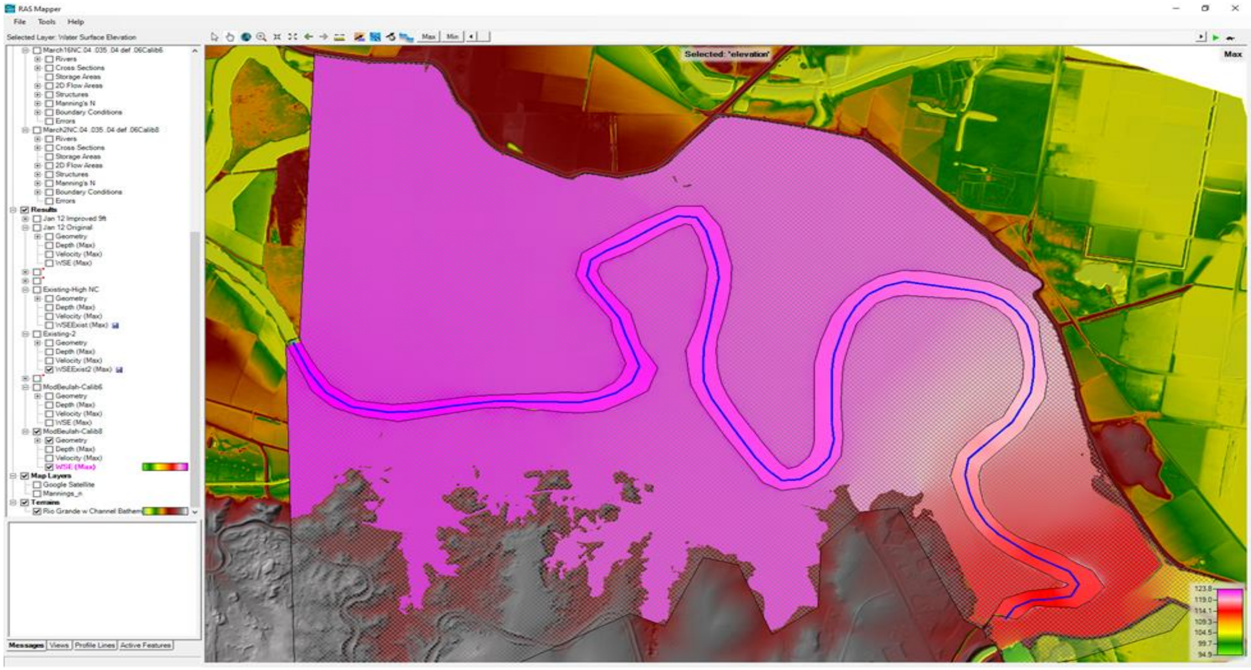


Figure A-1: Maximum Water Surface Elevation for Calibration Model

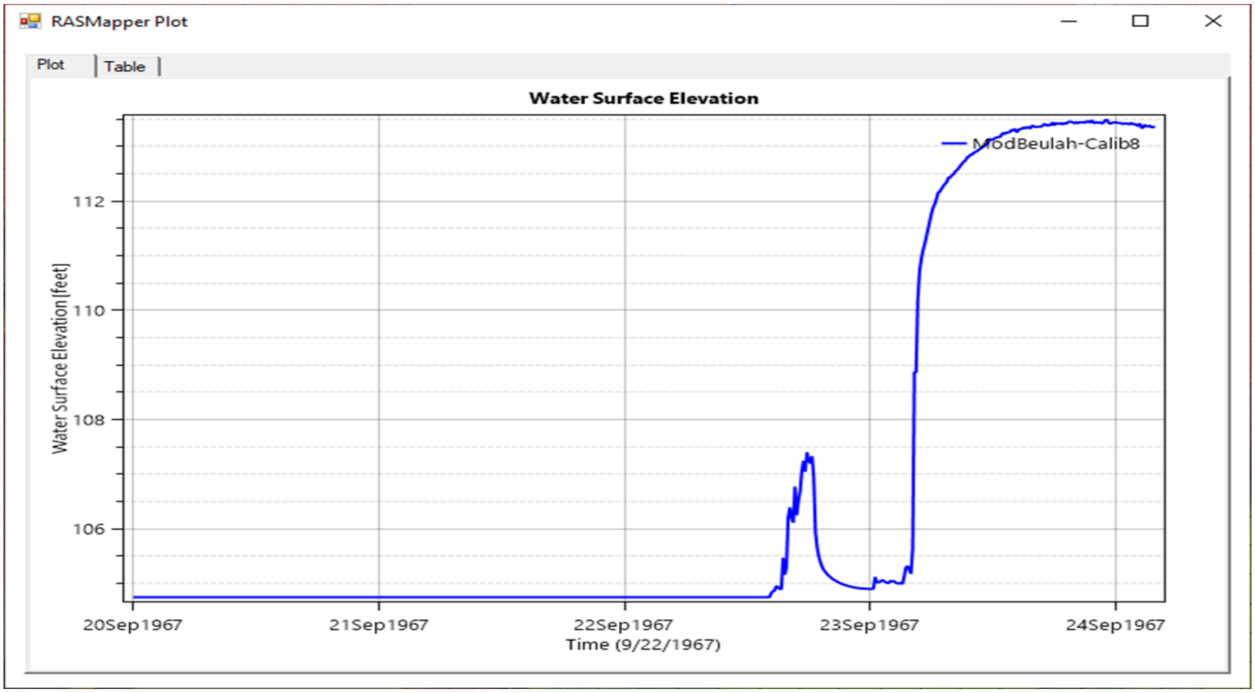
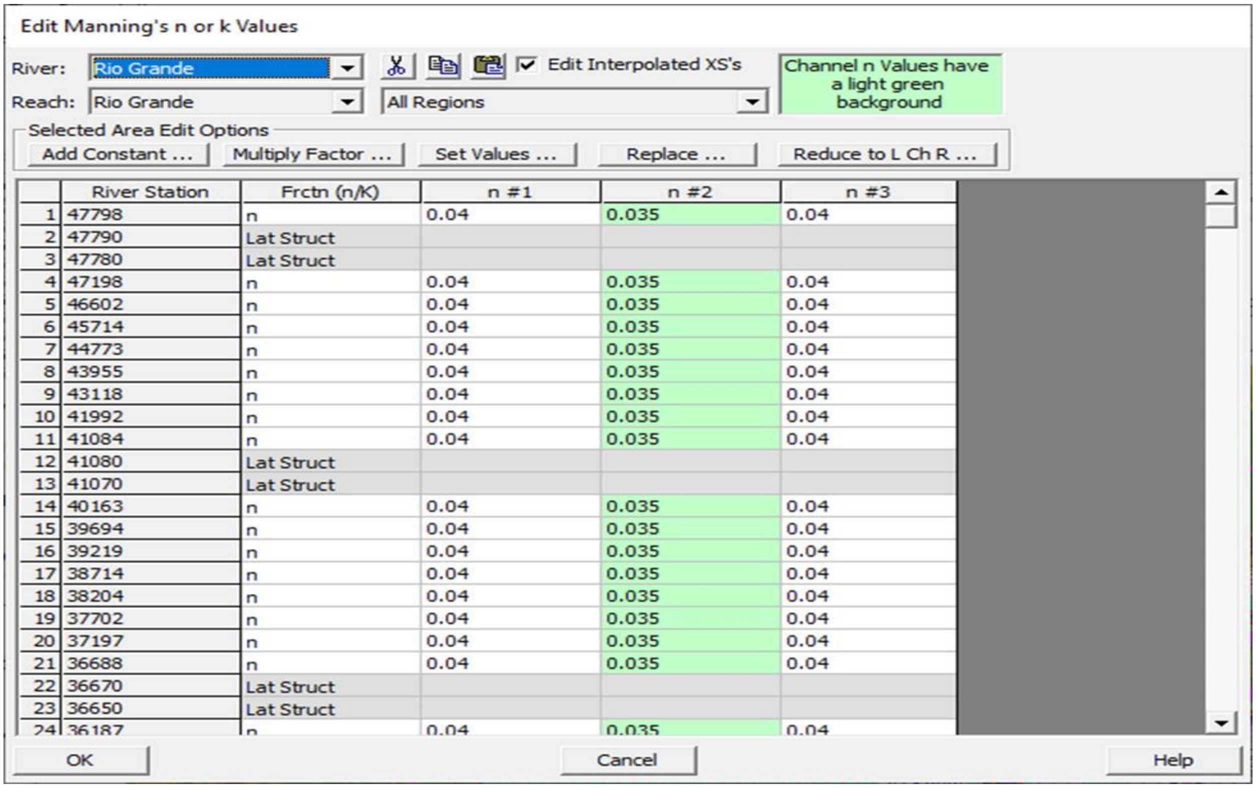


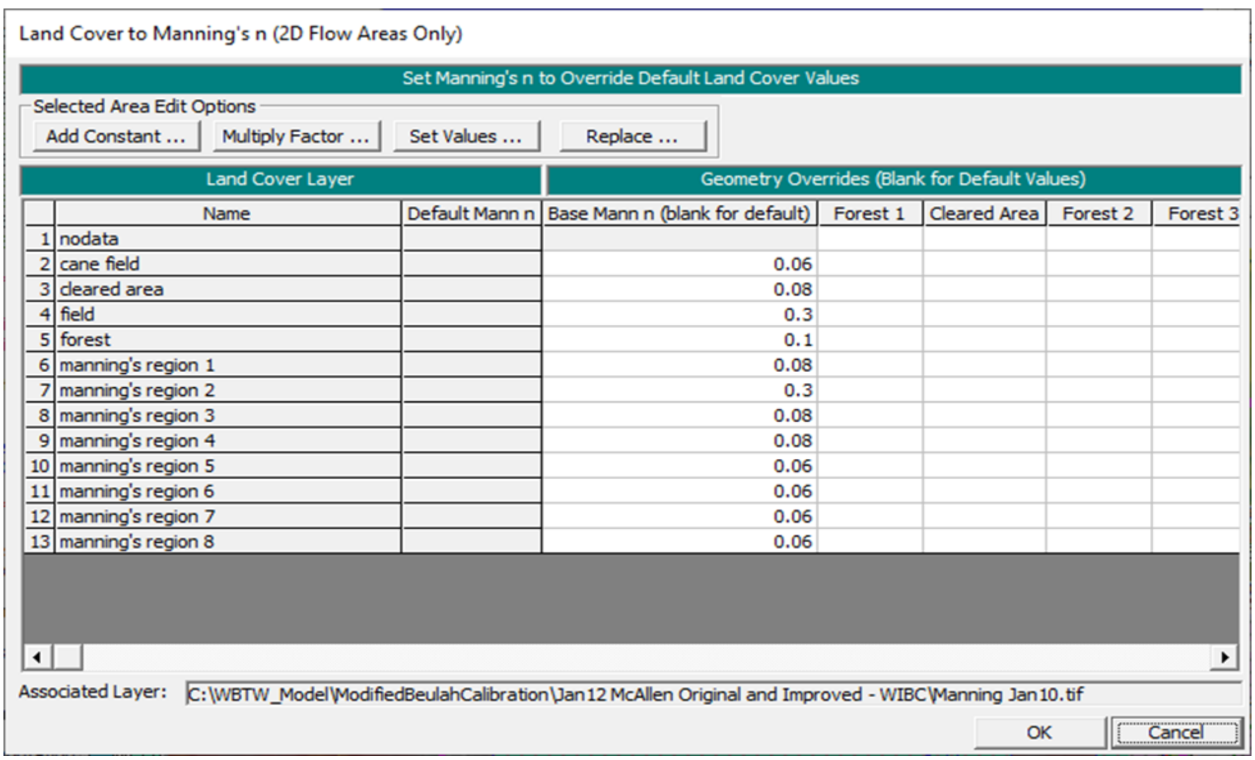
Figure A-2: Simulation of 113.45 Feet WSE 445 Feet Upstream of Anzalduas Dam During Calibration


Edit Manning's n or k Values
River: Rio Grande
Reach: Rio Grande
Edit Interpolated XS's
All Regions
Channel n Values have a light green background
Selected Area Edit Options
Add Constant ... | Multiply Factor ... | Set Values ... | Replace ... | Reduce to L Ch R ...

	River Station	Frctn (n/K)	n #1	n #2	n #3
1	47798	n	0.04	0.035	0.04
2	47790	Lat Struct			
3	47780	Lat Struct			
4	47198	n	0.04	0.035	0.04
5	46602	n	0.04	0.035	0.04
6	45714	n	0.04	0.035	0.04
7	44773	n	0.04	0.035	0.04
8	43955	n	0.04	0.035	0.04
9	43118	n	0.04	0.035	0.04
10	41992	n	0.04	0.035	0.04
11	41084	n	0.04	0.035	0.04
12	41080	Lat Struct			
13	41070	Lat Struct			
14	40163	n	0.04	0.035	0.04
15	39694	n	0.04	0.035	0.04
16	39219	n	0.04	0.035	0.04
17	38714	n	0.04	0.035	0.04
18	38204	n	0.04	0.035	0.04
19	37702	n	0.04	0.035	0.04
20	37197	n	0.04	0.035	0.04
21	36688	n	0.04	0.035	0.04
22	36670	Lat Struct			
23	36650	Lat Struct			
24	36187	n	0.04	0.035	0.04

OK | Cancel | Help

Figure A-3: Manning's Roughness Values for River Channel and Overbanks


Land Cover to Manning's n (2D Flow Areas Only)
Set Manning's n to Override Default Land Cover Values
Selected Area Edit Options
Add Constant ... | Multiply Factor ... | Set Values ... | Replace ...

	Land Cover Layer		Geometry Overrides (Blank for Default Values)				
	Name	Default Mann n	Base Mann n (blank for default)	Forest 1	Cleared Area	Forest 2	Forest 3
1	nodata						
2	cane field		0.06				
3	cleared area		0.08				
4	field		0.3				
5	forest		0.1				
6	manning's region 1		0.08				
7	manning's region 2		0.3				
8	manning's region 3		0.08				
9	manning's region 4		0.08				
10	manning's region 5		0.06				
11	manning's region 6		0.06				
12	manning's region 7		0.06				
13	manning's region 8		0.06				

Associated Layer: C:\WBTW_Model\ModifiedBeulahCalibration\Jan12 McAllen Original and Improved - WIBC\Manning Jan10.tif
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Figure A-4: Manning's Roughness Values for 2D Flow Areas

Appendix B: Maximum Water Surface Elevations

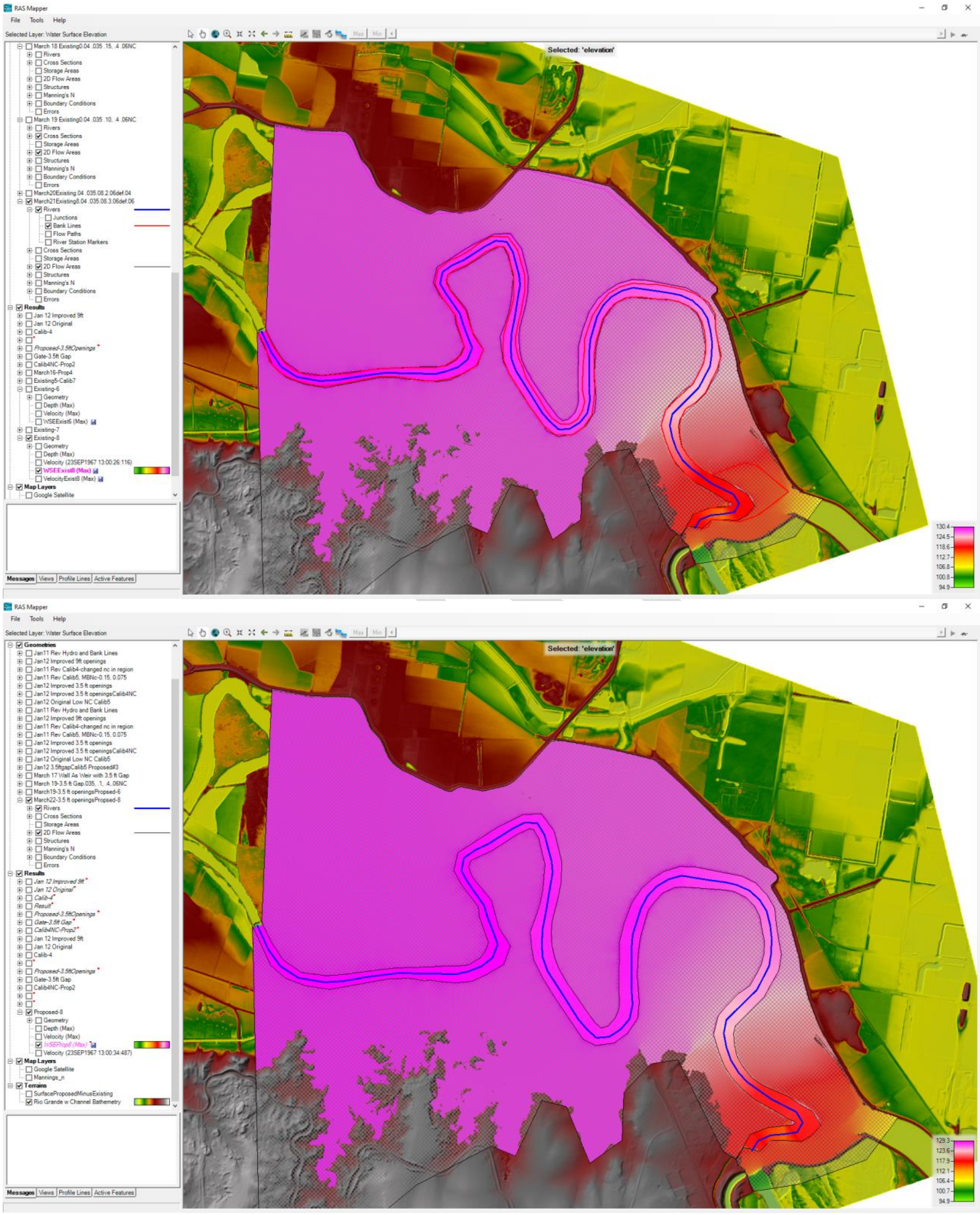

Figure B-1: Maximum water surface elevations for existing (top) and proposed model (bottom), proposed model elevations were slightly lower.

Appendix C: Flow Pattern-Velocity Distribution

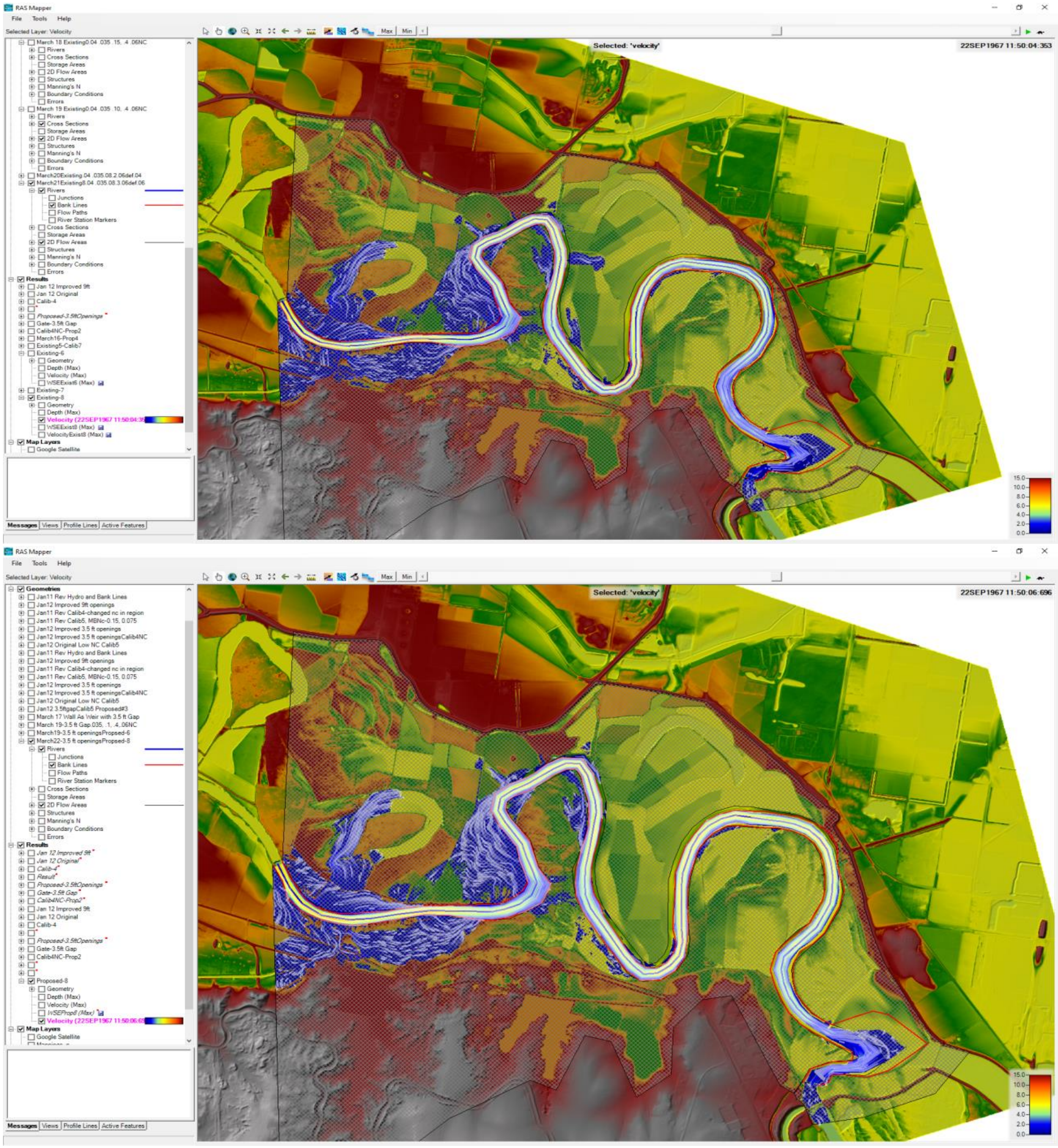

Figure C-1: Flow pattern on September 22, 1967 at 11:50 hours, existing (top), and proposed model (bottom); flow is blocked by fence at the bottom, so no flow in floodplain near the fence compared to existing condition model.

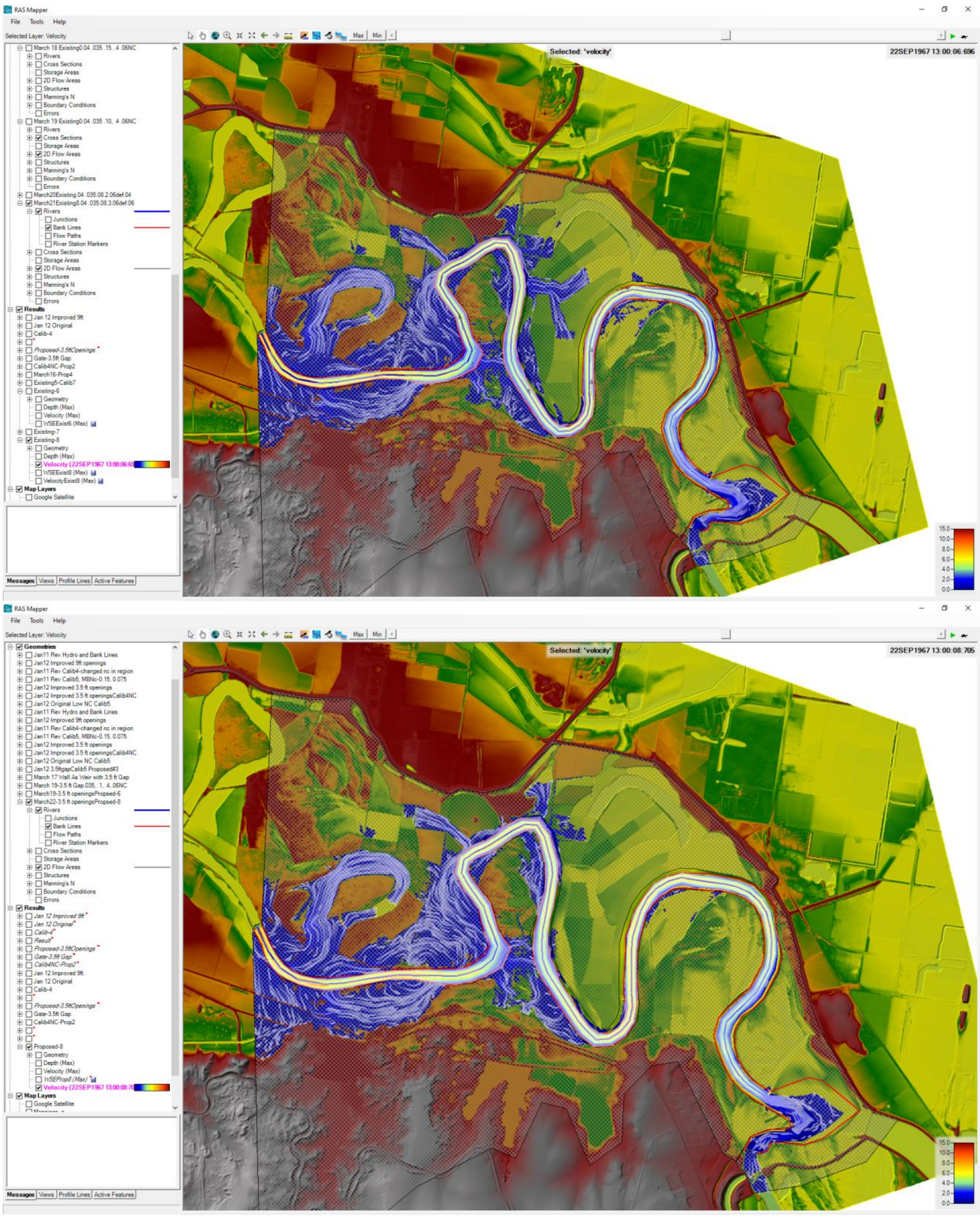

Figure C-2: Flow pattern on September 22, 1967 at 13:00 hours, existing (top), and proposed model (bottom); flow is blocked by fence at the bottom, so very little flow in floodplain near the fence compared to existing condition model. Flow in proposed model is going around the upstream end of the fence.

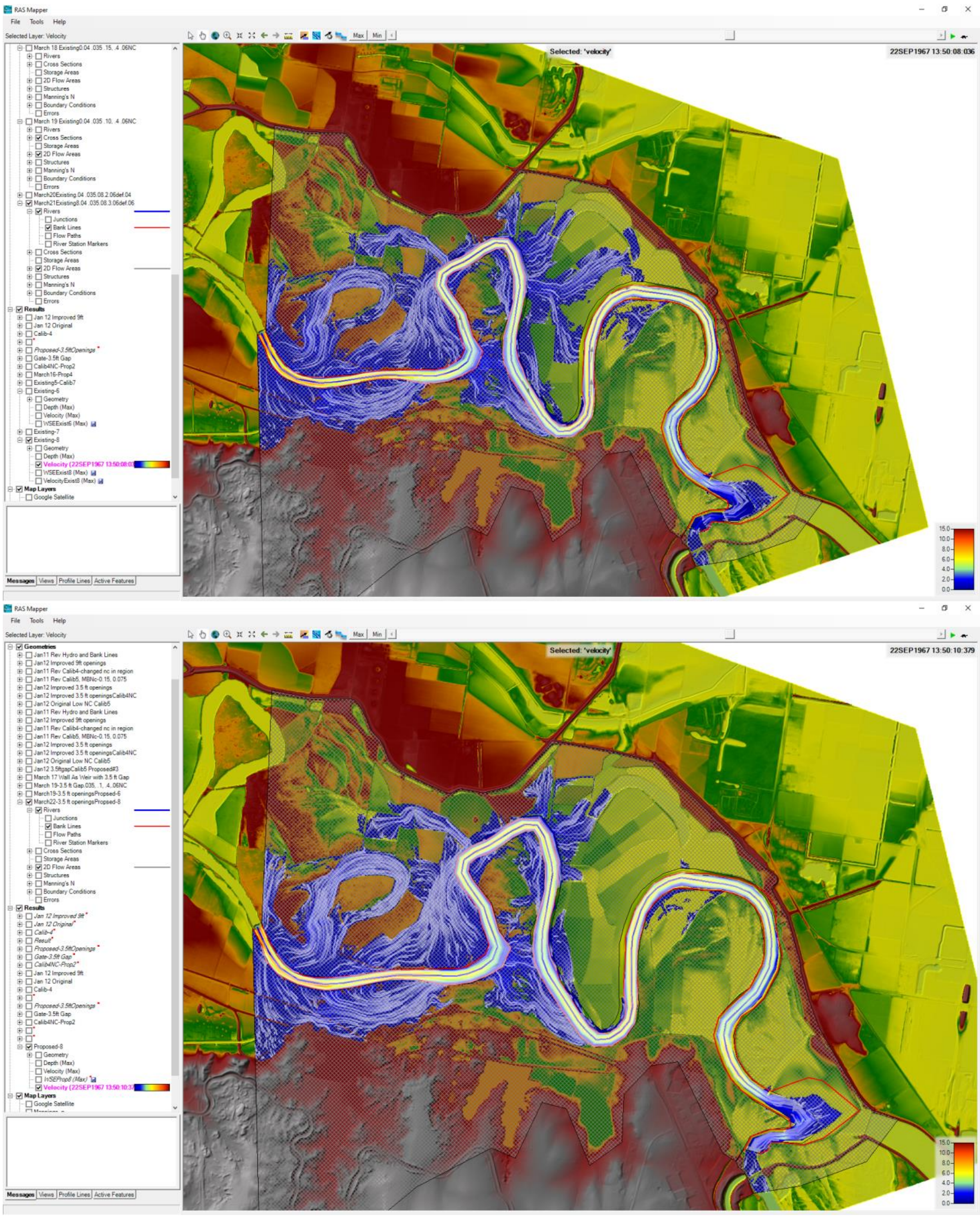

Figure C-3: Flow pattern on September 22, 1967 at 13:50 hours, existing (top), and proposed model (bottom); flow patterns are slightly different; more flow is in the floodplain near fence location in existing model compared to proposed model where there is resistance to flow from the fence (bottom).

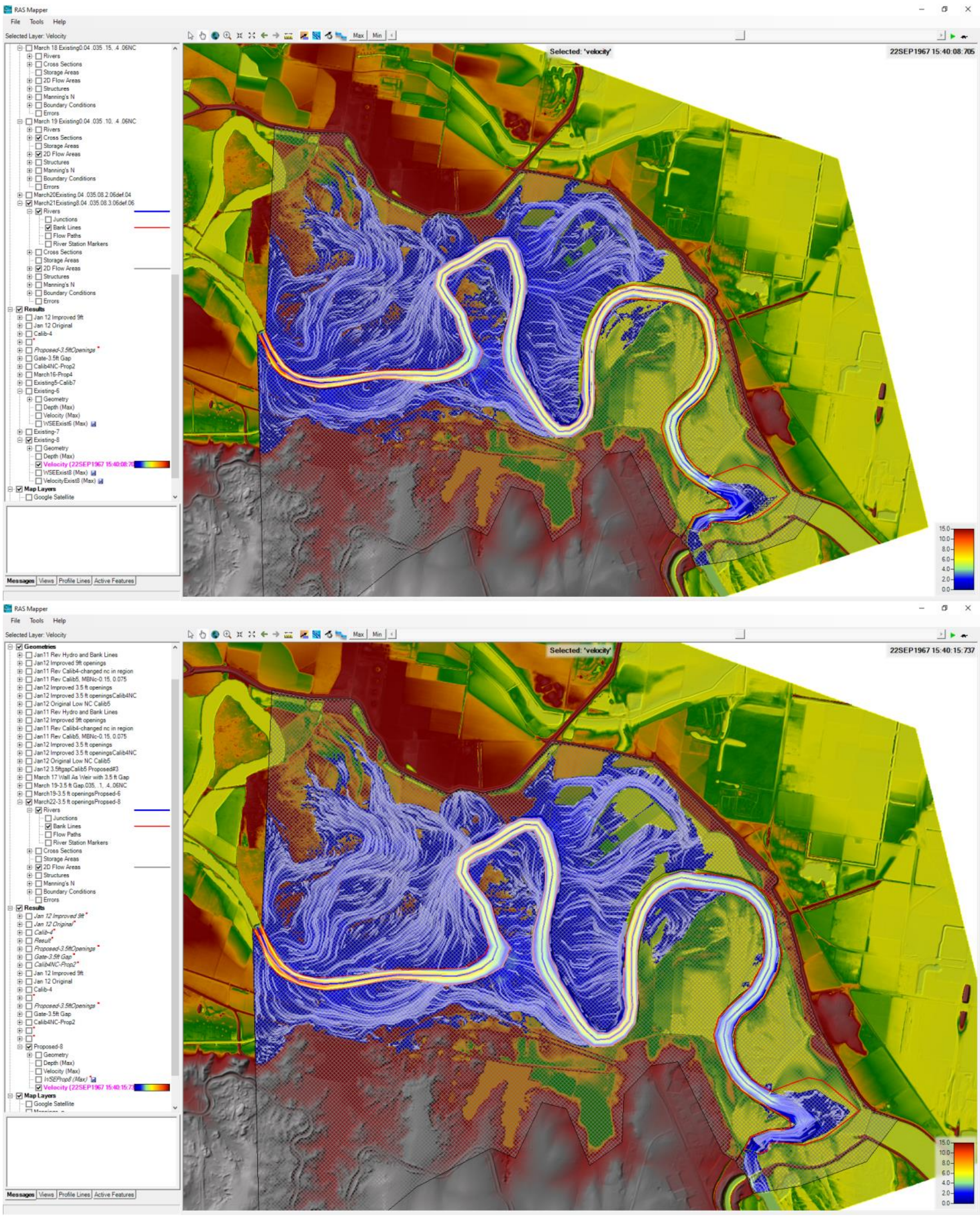

Figure C-4: Flow pattern on September 22, 1967 at 15:40 hours, existing (top), and proposed model (bottom); with high flows, flow patterns are similar for existing and proposed model near the fence location.

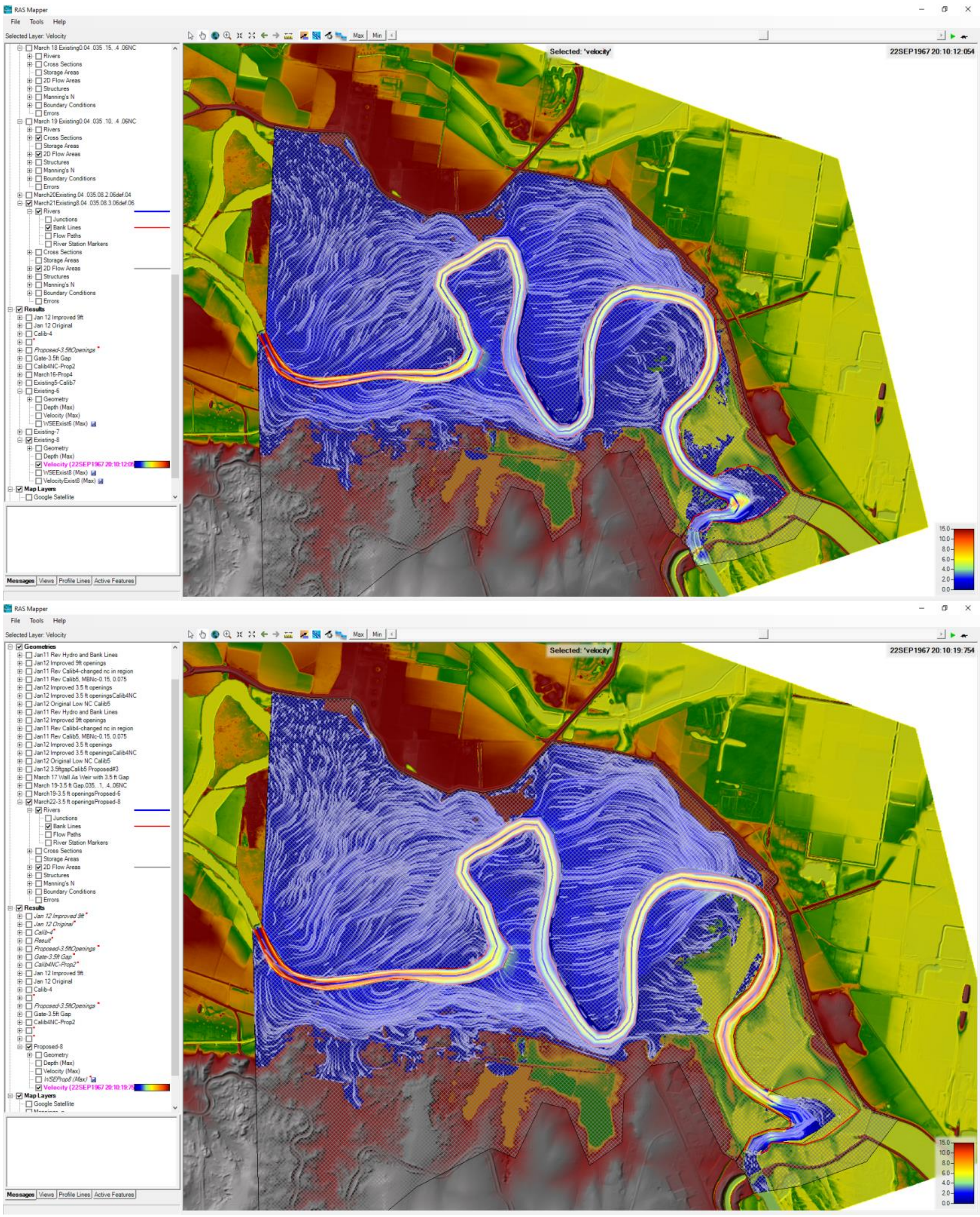

Figure C-5: Flow distribution on September 22, 1967 at 20:10 hours, existing (top), and proposed model (bottom); with high flows, flow patterns are similar near the fence location for existing and proposed model; flow patterns are slightly different at the downstream areas.

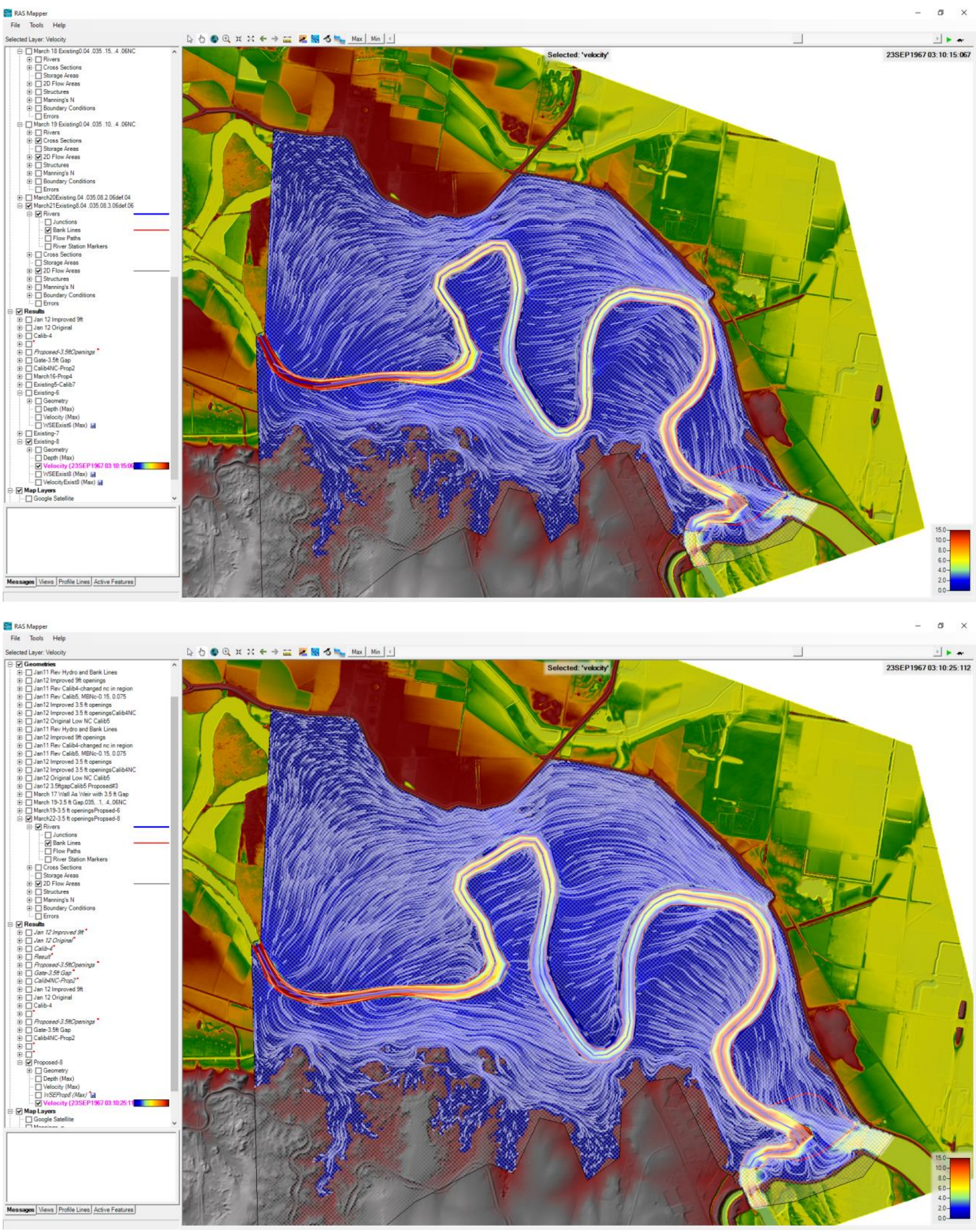

Figure C-6: Flow pattern on September 23, 1967 at 3:10 hours, existing (top), and proposed model (bottom); with peak flow, flow patterns are similar for both existing and proposed model; flow can go through, around and over the fence for proposed model.

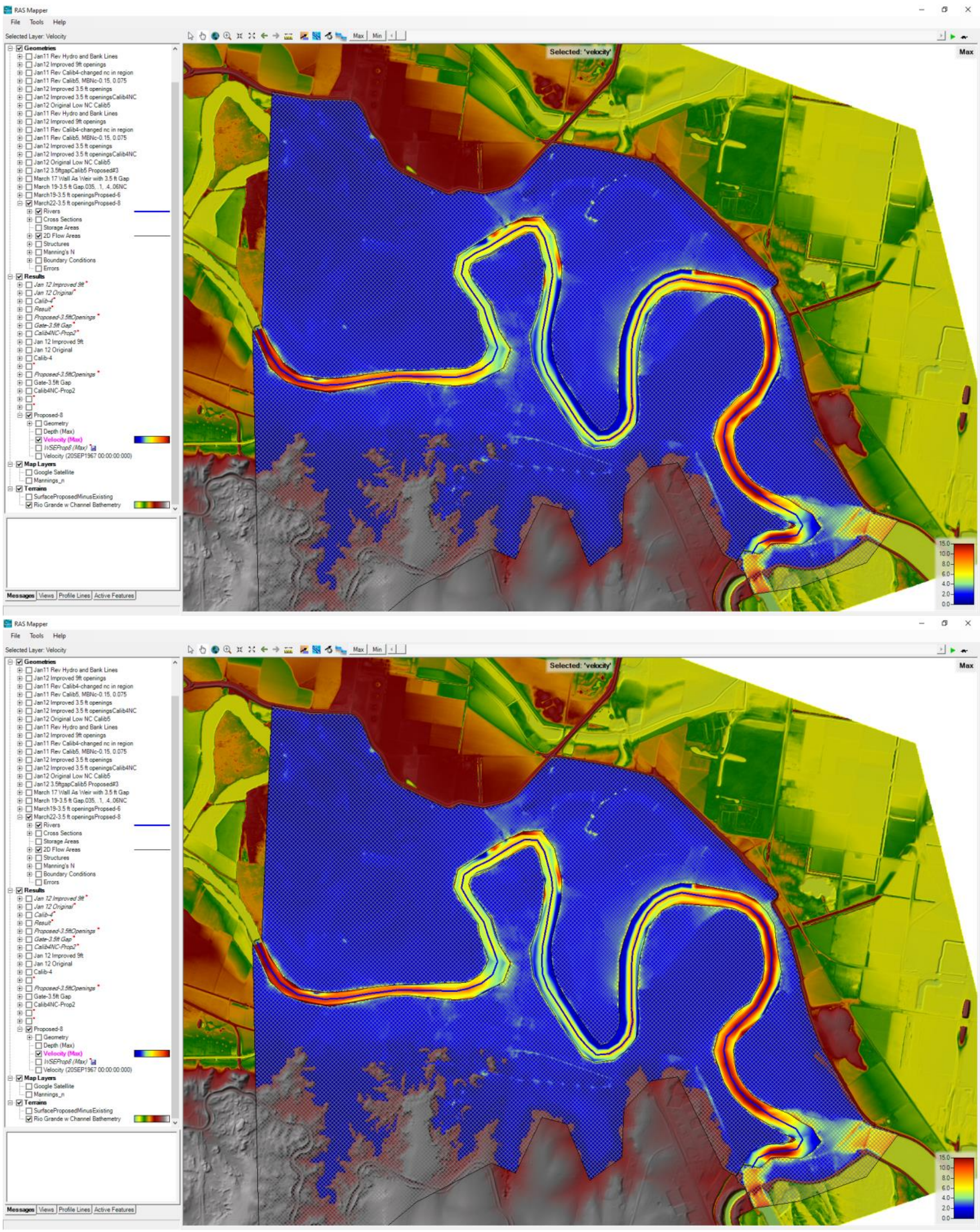

Figure C-7: Maximum flow velocities for existing (top), and proposed model (bottom); with high flow, both models exhibited erosive velocities along the river channel.

HEC RAS UNSTEADY FLOW HYDRAULIC MODEL

DETAILED SECTION AT MISSION TEXAS FOR

BORDER FENCE INSTALLATION

Supplement to the Lower Rio Grande Flood Control Project (LRGFCP)

Submitted by TGR Construction, Inc

For construction from River Station (RS) 172.5 to 175.5

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STATE OF TEXAS
TIMOTHY ALAN STARK
117381
LICENSED
PROFESSIONAL ENGINEER

December 31, 2019

Registered Professional Engineer (Civil)
CERTIFICATE NO.
22183
GREG H.
GENTSCH
Date Signed 12/31/19
ARIZONA, U.S.A.

TABLE OF CONTENTS

Introduction

This analysis is being submitted to identify and address the potential effects of the construction of a new bollard fence within the floodplain on the U. S. side of the Rio Grande from River Stations (RS) 172.5 to 175.5. The Project is located between Reynosa, Tamaulipas and McAllen, Texas. This site is located upstream from the Anzalduas Dam in Hidalgo County, Texas. The location is shown with the green place-mark in Figure 1 below.

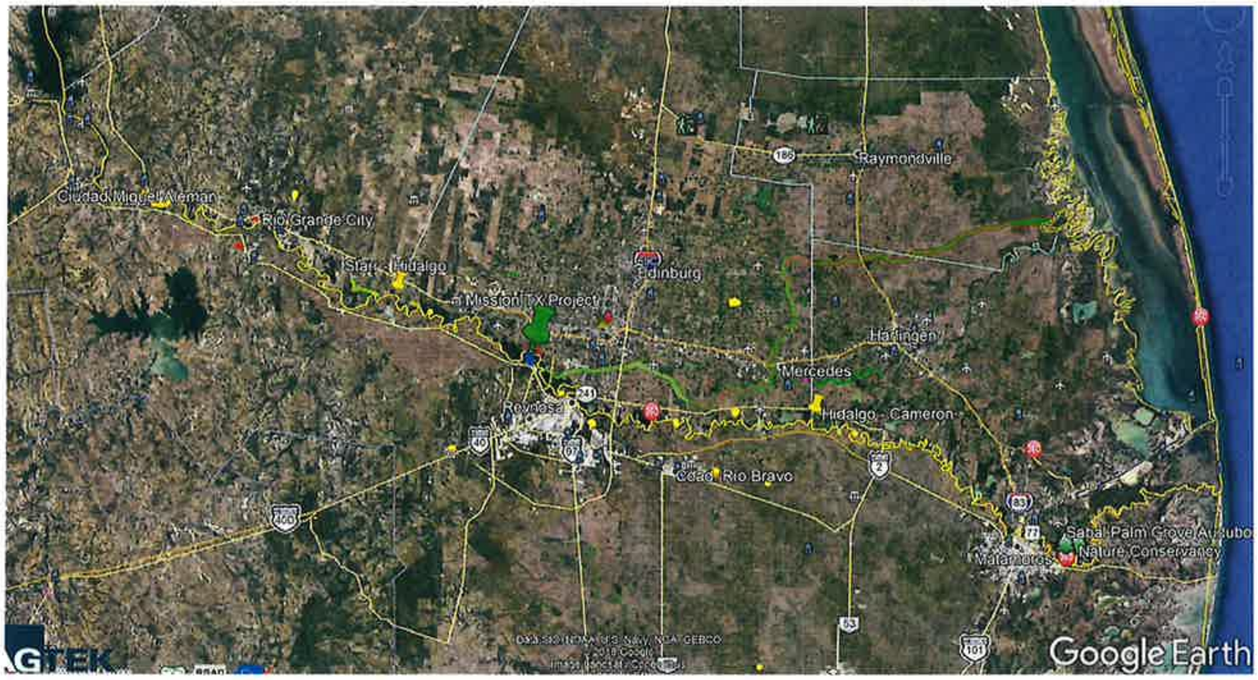

Figure 1. Area map of Hidalgo County & Tamaulipas State at Rio Grande/Rio Bravo

Photo from Google Earth

The proposed fence would be constructed on private land which is in the floodplain. This land is also completely within the active reservoir area created by the operation of Anzalduas dam. The fence alignment will run parallel to the river and will be offset from the bank at least 35 feet. All of the existing tall vegetation on the bank will be replaced with deep rooted grass. All invasive vegetation is being removed and the banks are reshaped to a 5 to 1 or flatter slope. This will reduce the potential for erosion by eliminating caving banks as well as preventing trees and other vegetation from falling into the river channel during periods of higher flows when the reservoir is rising or receding. The fence alignment is shown as the blue line in Figure 2 below.

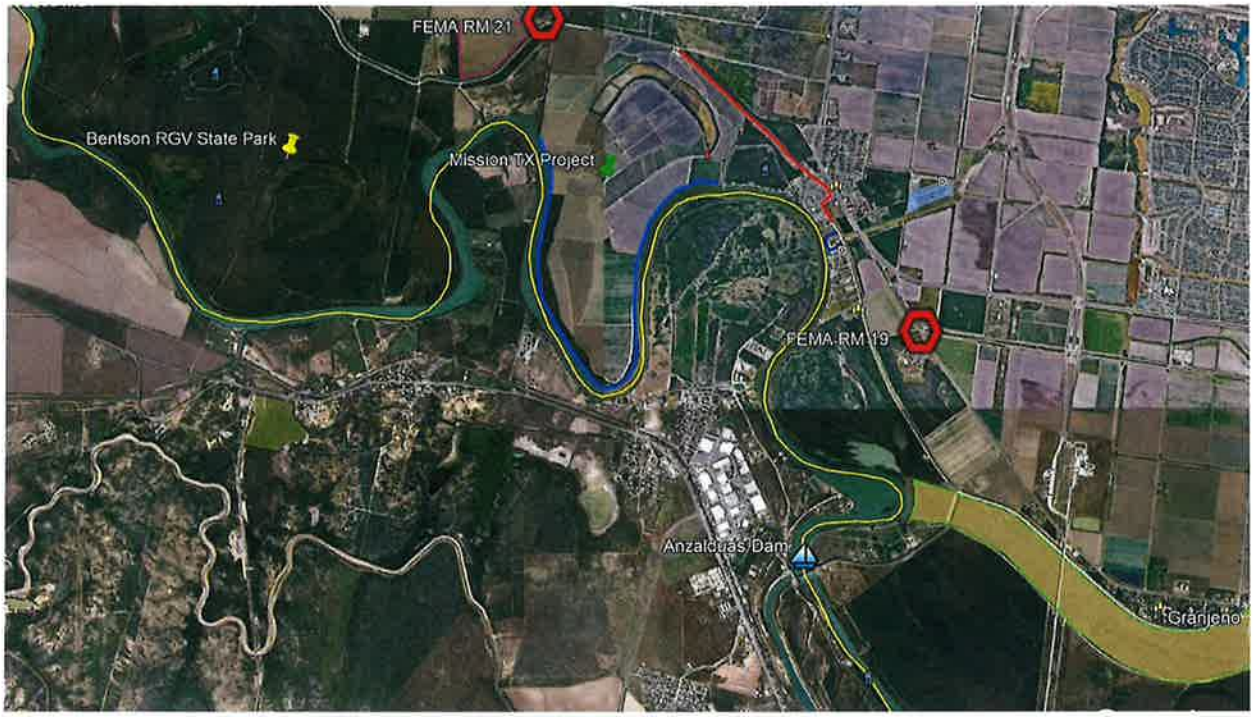

Figure 2. New Bollard Fence RS 172.5 to 175.5

Photo from Google Earth

Description

The design maximum flood used in the previous studies is 235,000 cubic feet per second (CFS) and the reservoir water surface was determined by those studies to reach a maximum pool elevation of 121.16 near the dam and up to 126.85 at RS 174.6. The overall velocity of the water body at that stage would be relatively low as it moves towards the Dam and Main Channel weir due to the large cross sectional area. There are no drainage structures or highway bridge crossings in the area or immediately upstream or downstream of the fence construction. There are no large tributary inflows within the project limits. There are no points of divergent flow above RS 170.02 at the weir for the main spillway, which is at elevation 106.1. The only prior point of divergent flow was the Mission Inlet, which is now closed by a levee. This project is designed to allow sheet flow to evenly flood the fields on this site as the river rises.

There is one power line on the property and 3 pumps. One small open sided structure is a concrete pad supporting two irrigation pumps on the bank of the river at RS 173.08. The floor level of this pump house structure would begin to become submerged in cases where the reservoir pool rises to elevation 107 or higher. A photo of this pump house is included below.

Figure 3. Photo of Irrigation Pumps at RS 173.08

This detailed study area does not include any of the downstream floodway channels since they are all separated from this particular reach of the Rio Grande by the operation of Anzalduas Dam and therefore will have no significant backwater effects on the elevations in this reservoir.

A key part of this project is the stabilization and protection of the existing river bank. This will require an ongoing maintenance program. As previously noted, the existing caving banks will be regraded to a configuration with a 5 to 1 slope or flatter that can then be supported by normal maintenance and mowing. The removal of the invasive species, Carrizo Cane (*Arundo donax*), is being done mechanically, without using chemicals. The eradication of *A. donax* is supported by the Texas State Soil and Water Conservation Board (TSSWCB) which identifies it as "...one of the greatest threats to the health of riparian ecosystems in the southwestern United States." [1]

The Carrizo Cane has a relatively shallow root system which creates a condition where caving banks are more prevalent after the invasive plants have displaced the deeper rooted but slower growing grasses. There are many examples of improved bank protection on both sides of the river that demonstrate how well maintained and more stable grass cover will improve the bank.

Figure 4. Upstream area near the Main Floodway weir (looking downstream).

Well maintained grass cover to prevent scour.

In addition, 70 trees are being planted along the bank to improve stability during higher flows. These will fit in with the dozen or more existing palm trees that were well established at the water's edge. These will be Bald Cypress (*Taxodium distichum)* and Texas Sabal *(Sabal texana).* Our initial clearing and mapping activities also found dredged material piled on the left bank from RS 174.4 to RS 174.7 at an elevation of 114 to 116.5. In addition, this area also had a very broad expanse of thick vegetative cover averaging 320 ft. wide. This elevated and overgrown area would reduce the amount of lateral flow to the left overbank and across the private land. The final grade will be constructed at elevations 111 to 112 and all flow through the 5" wide gaps in the bollard fence will be unrestricted above elevation 112. There are no panels or other horizontal members incorporated in this fence design. Due to the parallel alignment of the fence and lower overbank velocities very little floating debris should be caught in the fence. The foundation will be reinforced concrete. An adjacent parallel concrete road will also protect it.

The HEC RAS input reflects the most current information from our on-site topographic survey performed in conjunction with normal clearing operations prior to our fence construction.

All other supporting information was extracted from a prior LIDAR mapping operation which was flown in 2011. The data files were received in our offices on December 18, 2019 via an external storage drive sent to Fisher Industries by IBWC.

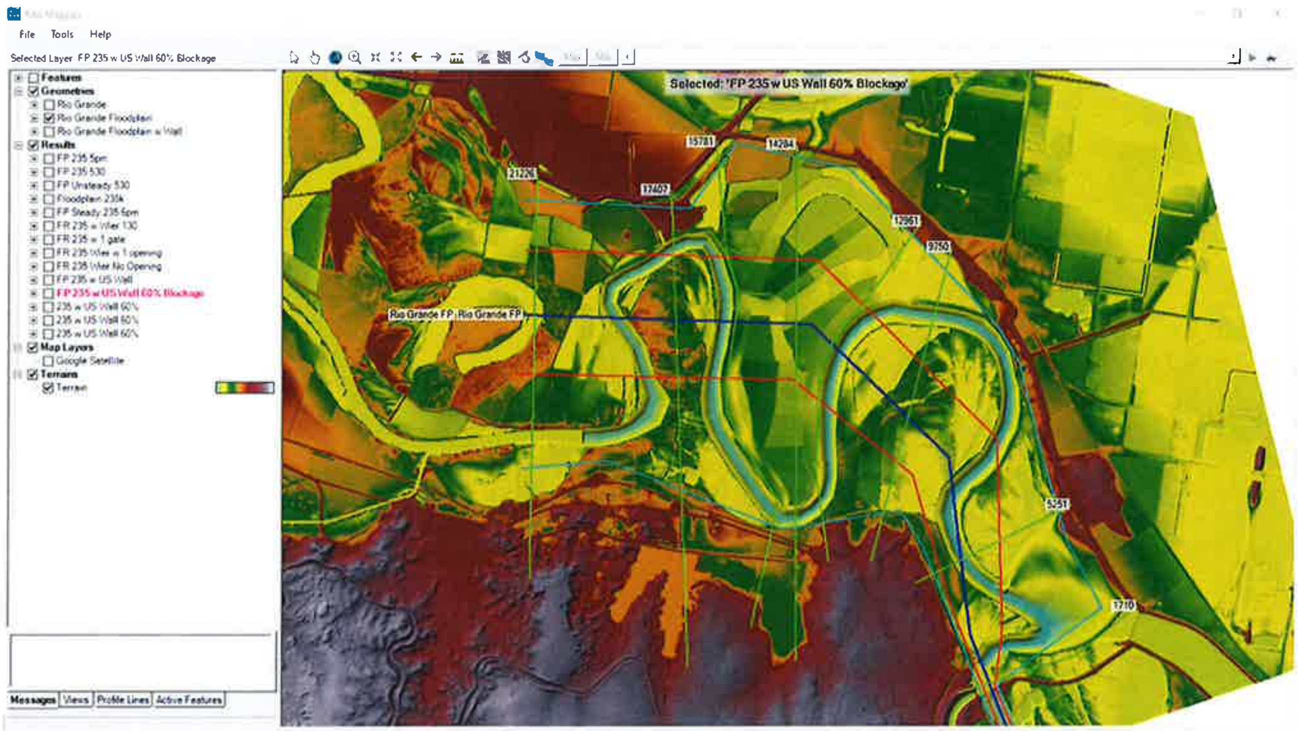

Figure 5. Study Area RS 170.0 to 177.25

Photo from RAS Mapper (Note stations for reservoir in feet from Anzalduas Dam differ from RS)

The lateral flow area is essentially the footprint of the reservoir in the Project study area. This is generally shown in figure 5. The cross section spacing on the main river channel was set at 100 feet for the full length of the work area to accurately depict the location of the bollard fence and the improved left bank. Then eight major cross sections were also taken in an orientation perpendicular to the general direction of flow in the reservoir. These are shown with labels in Figure 5 above. It is clear that all features on this property can become submerged. No occupied structures will be constructed. There are no additional levees or restrictions to fluctuations in the area of the reservoir. Maintenance of the 5 to 1 slope as well as the edge of the river bank will be extremely important. In situations where any erosion occurs it should be repaired in a timely manner and the area reinforced if necessary using riprap rock with an 18" to 36" diameter or an alternative geo-cell slope stabilization material. All banks will be easily visible to monitor and simplify maintenance. Law Enforcement will have access to the roadway.

The profile shows the new fence in reservoir cross section 15770 which is on the western side of the project site. Note the ground elevation will be 112 upstream of the small transition and 111 downstream of that point. This transition occurs at RS 174.4 on the main channel.

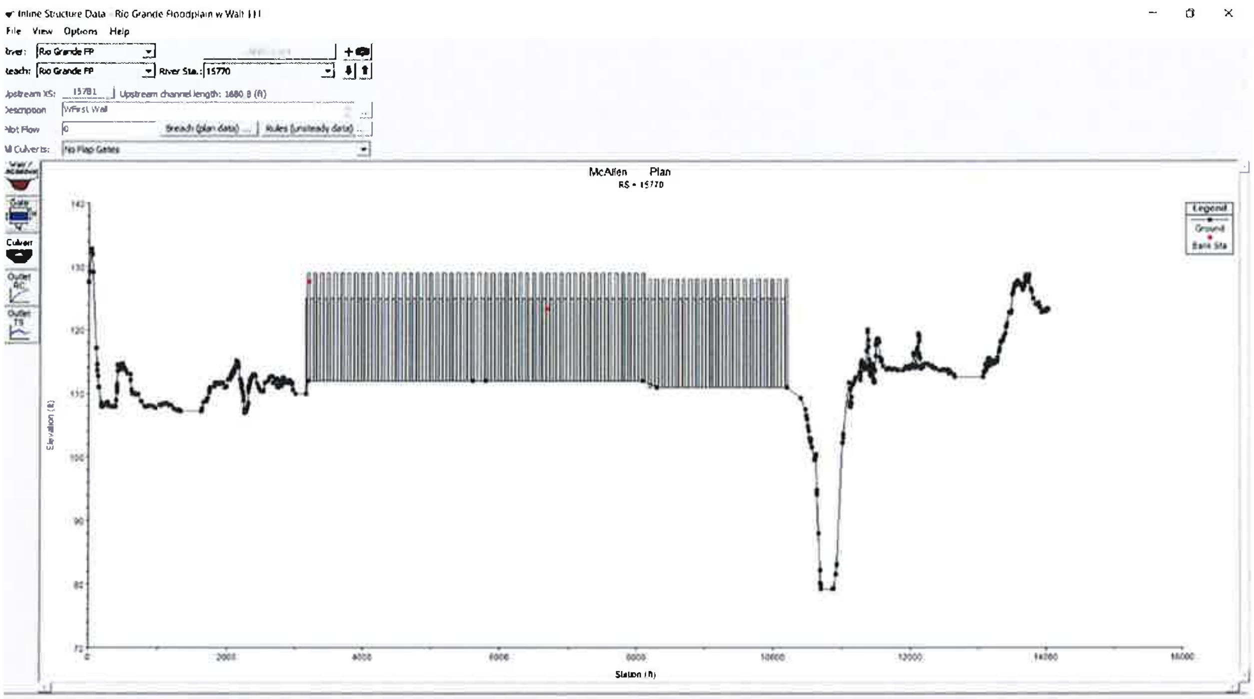

Figure 6. Profile 15770 (reservoir station in feet upstream from Anzalduas Dam)

North Bypass Area

The lowered sill at the upstream end of the fence alignment is designed to allow larger volumes of water to enter the adjacent field early and then flow into the oxbow lake at the north end of the property. This area will be constructed with a top elevation of 110 where it ties into the concrete roadway. This area is shown above label RS 175.4807 in upper left section of Figure 7. The inundation maps shown in Appendix C also show that the old oxbow lake and all of the lower fields in this area initially fill from the lower areas to the east and at the eastern property boundary. During the early stages of a large flood event the water surface (W.S.) elevation in this reservoir begins to increase as the retained volume inundates lower areas that are adjacent to and connected with the channel bank. Some flow will then begin to move across the fields when the W. S. in the channel reaches 110. This allows the even distribution of water over the field as the reservoir fills with inflow from downstream at elev. 108 and above as well as at this point. The even progression of the inundation on both sides of the fence will ensure that the velocity will remain relatively low in these areas. See velocity distributions in Appendix D.

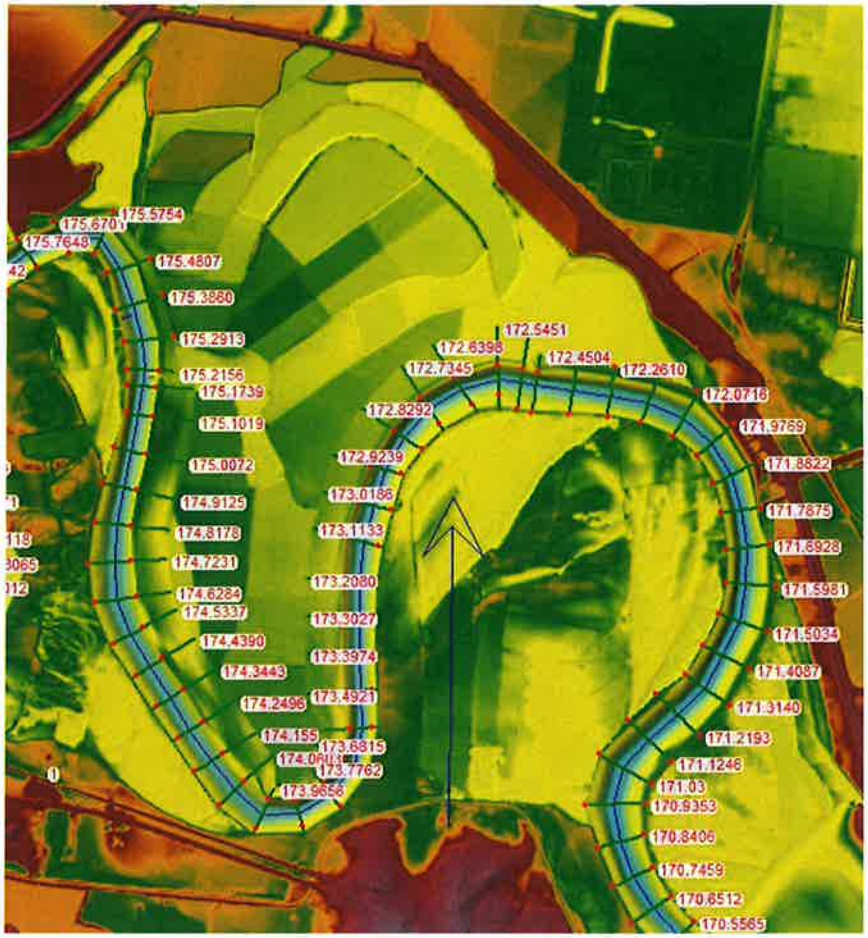

Figure 7. Existing North Bypass area elevations will allow early inundation (see Appendix C)

Bollard Fence

The placement of the bollard fence along the perimeter of this site is designed with a wider gap to allow more volume to pass. The bollards are also turned at 45° for an improved inlet condition. This fence will be less of an obstruction to lateral flows than the heavy expanse of vegetation which has been removed from this same area. The regrading of this area and the removal of the dredging spoils which were encountered will result in less overall resistance to any lateral flow in this segment. Lateral flow through the gaps in the bollard fence will therefore have a lower velocity when the level of the reservoir is considered by the time the main channel flow reaches an elevation of 112. The profile elevation at the base of the fence will be 111 from RS 172.5 to RS 174.4 and 112 from RS 174.4 to 175.5. The grading creates a uniform sloped bank that is lower and less susceptible to erosion. As the reservoir fills to the maximum pool elevation the storage will maintain relatively equal levels on both sides of the new fence and lateral velocities between the Floodway and our site are lower as noted.

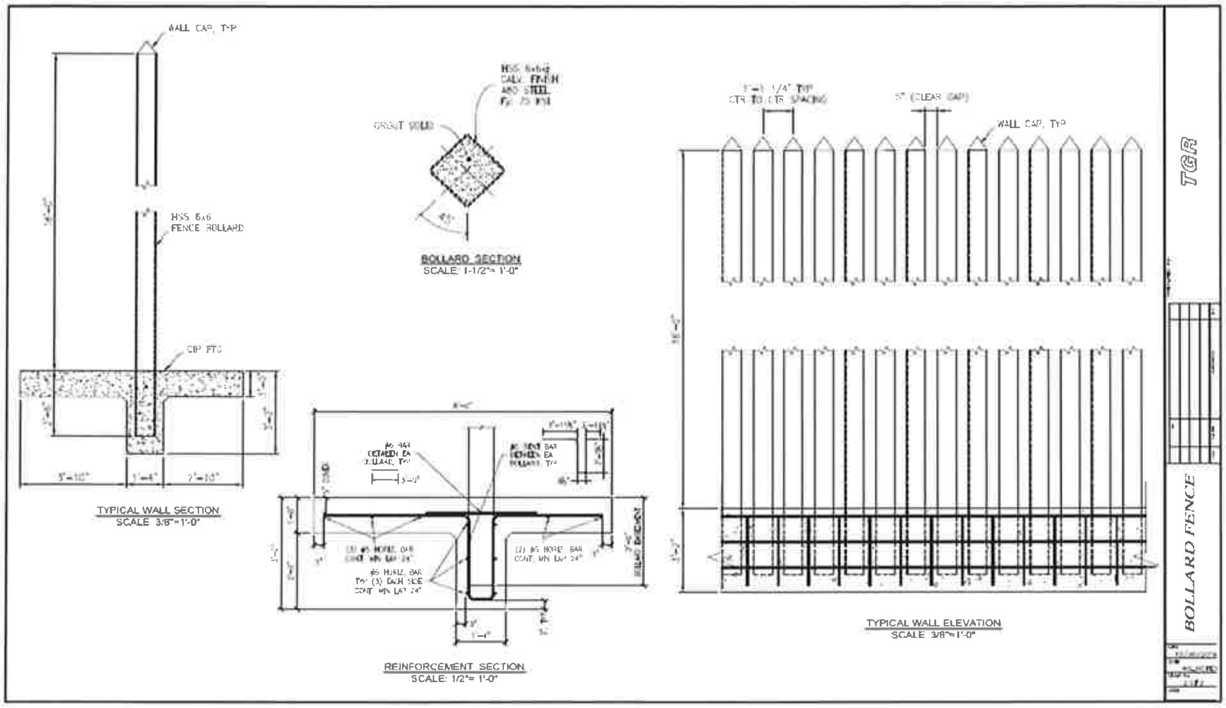

Figure 8. Detail of Bollard Fence

Evaluation of HEC RAS output

The reservoir flow was modeled with cross sections at a varied spacing averaging about 3,000 feet. The first application was to address the existing condition with the mapping to include the heavier vegetation on the west bank of the property. The second run was modified to represent the cleaned and sloped bank as well as the removal of the overgrown areas and replacement by the line of bollards. The alignment of the bollard fence is given by the blue line shown in figure 2 on page 2. The 235,000 cfs design hydrograph provided in previous IBWC reports will be used as the upstream boundary condition for these two unsteady 2D flow analyses (see Appendix E).

The HEC RAS report indicated that the water surface (W.S.) elevations were not quite as high as previously indicated in the 2008 report. The overbank velocities are in the expected range. Some improved flow efficiency can be expected due to the proposed 5:1 bank condition with significantly less vegetation. The bollard fence was first modeled with 0% obstruction as 100 foot long panels with a 50 foot long opening. This is an estimated representation of the hydraulic capacity of the bollards when oriented at 45° to provide a better inlet condition (see fig. 8). The final analysis was run with 30% obstruction as recommended by IBWC. This was represented with 100 foot long panels with a 40 foot long opening. There was no significant difference due to the manner in which this reservoir floods over the project site with the early portion of the lateral flow event entering the project site from the east. The relief provided by the north bypass channel will allow Left Overbank sheet flow to begin when the water surface

exceeds elevation 110 at RS 175.5 as previously noted. The reservoir level will also be at 111 or higher by the time the lateral flow begins to cross the alignment of the bollard fence. The gradient and the velocity will also be minimal when the reservoir water surface reaches an elevation of 108 and first begins to inundate these fields from the east side of the peninsula. See appendices C-1 & C-2 for comparison of different stages of inundation occurring in this reservoir as the unsteady design flow event occurs. C-1 shows the existing condition. C-2 shows the stages of inundation with the fence in place. Also see appendices D-1 & D-2 for comparison of the flow velocities in this reservoir as the unsteady design flow event occurs. D-1 shows the existing condition. D-2 shows the flow velocities with the fence in place.

River Station (mi)	W. S. Elev. Before	W. S. Elev. After	W. S. Elev. – 2008 report
176.8	124.51	124.52	128.15
174.6	123.92	123.99	126.85 (127.85) typo?
174.2	123.81	123.90	127.16
173.9	123.65	123.75	n/a
171.8	123.04	123.07	126.4
170.8	122.22	122.22	125.1

Example of Unmitigated bank instability

The two photos on the following page (figs. 9 and 10) show an example where a large event significantly eroded a bank in this same reservoir. It is noteworthy that the combination of the constricted river channel and the exposed vertical edge of the bank created the condition where significant erosion would occur. A complimentary and coordinated maintenance effort to establish gently sloped and protected riverbanks while taking every opportunity to eradicate the invasive Carrizo Cane will go a long way towards keeping the alignment of the river in the correct and mutually agreed upon location. This is a key issue with regard to protecting the river and the overbank areas from further damage and degradation. Figure 9 at the top of the following page shows the prior condition in 2009 with a relatively narrow channel (due to accretion on the inside of the bend) and a vertical exposed bank on the outside of the bend. Figure 10 at the bottom of page 10 shows the eroded area as photographed later in 2013.

Figure 9. Rio Grande 2009

Figure 10. Rio Grande 2013

Conclusions and Recommendations

- The bollards will take up no appreciable volume in the Anzalduas reservoir and do not significantly impede the movement of the water as the reservoir fills and draws down.
- Maintenance is necessary to establish and support a specific type of vegetation on river banks to support deep rooted grass species and eradicate invasive Carrizo Cane. Maintenance must keep the invasive species under control.
- There is no significant deflection due to the improved inlet conditions at all of the openings in the bollard fence. The flow pattern calculated for the downstream movement of the reservoir shows that all of the impinging flow passes through the fence with a rise of approximately 0.1 feet or less.

Credits

[1] Excerpt from page 1, paragraph 3, Rio Grande Carrizo Cane Eradication Program by Texas State Soil and Water Conservation Board. Version 20161014.

Appendix List

Appendix A. Rio Grande Carrizo Cane Eradication Program by Texas State Soil and Water Conservation Board. Version 20161014.

Appendix B. Left bank Cross sections from Fisher on-site survey; RS 172.5 to 175.5.

Appendix C-1. Screen shots from program - progression of inundation (existing condition).

Appendix C-2. Screen shots from program - progression of inundation (final condition).

Appendix D-1. Screen shots from program - velocities as reservoir fills (existing condition).

Appendix D-2. Screen shots from program - velocities as reservoir fills (final condition).

Appendix E. Design hydrograph used for unsteady 2D flow.